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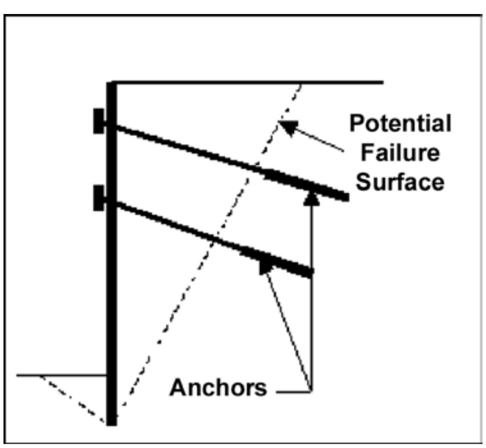
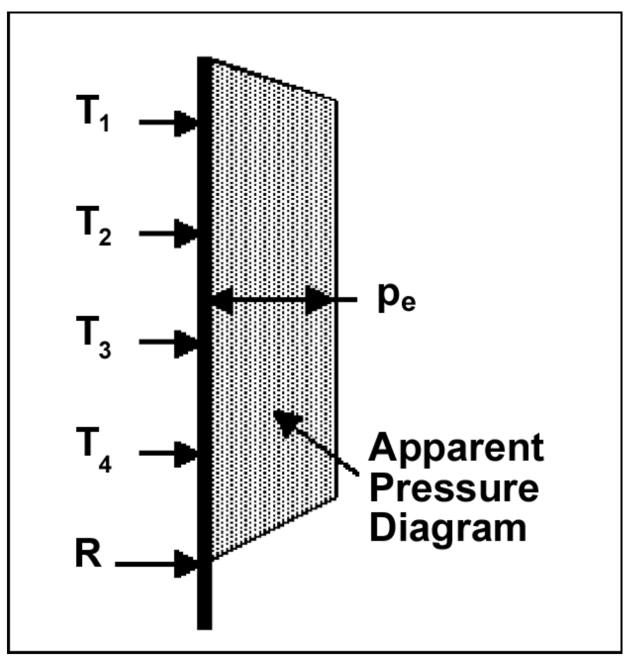
Simplified Procedures for the Design of Tall, Flexible Anchored Tieback Walls

Robert M. Ebeling, Muluneh Azene, and Ralph W. Strom

November 2002





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ABSTRACT: In practice, the procedures used to design flexible tieback wall systems differ from those used to design stiff tieback wall systems. In the design of flexible tieback wall systems, apparent pressure diagrams are commonly used to represent the maximum loads the tieback wall system might experience during construction. Apparent pressure diagrams used in an equivalent beam on rigid supports analysis are demonstrated in this report. Analyses are performed for flexible wall systems in both cohesionless and clay soil. Flexible wall systems include a soldier beam–wood lagging system and a sheet-pile system. Wall heights of 25, 35, and 50 ft (8, 11, and 15 m) are evaluated.

Apparent pressures are developed on a "total load" approach using limiting equilibrium procedures. Apparent pressure diagrams are nonsymmetrical in shape, as recommended in FHWA-RD-97-130 ("Design Manual for Permanent Ground Anchor Walls," Federal Highway Administration).

Designs are provided for two performance objectives: "safety with economy" and "stringent displacement control." A factor of safety of 1.3 is used for the safety with economy designs for which displacement control is not a significant concern. A factor of safety of 1.5 is used for the stringent displacement control designs, for which it is assumed that displacements must be minimized to prevent settlement-related damage to nearby structures.

Comparisons are made between the safety with economy and the stringent displacement control designs for the wall heights indicated above.

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Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic inches	16.38706	cubic centimeters
degrees (angle)	0.01745329	radians
feet	0.3048	meters
inches	25.4	millimeters
kips (force)	4.448222	kilonewtons
kips (force) per square inch	6.894757	megapascals
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
pounds (mass) per square foot	4.882428	kilograms per square meter
square inches	6.4516	square centimeters

Preface

The study described in this report was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Innovations for Navigation Projects (INP) Research Program. The study was conducted under Work Unit (WU) 33272, "Soil-Structure Interaction Studies of Walls with Multiple Rows of Anchors."

Dr. Tony C. Liu was the INP Coordinator at the Directorate of Research and Development, HQUSACE; Research Area Manager was Mr. Barry Holliday, HQUSACE; and Program Monitors were Mr. Mike Kidby and Ms. Anjana Chudgar, HQUSACE. Mr. William H. McAnally of the Coastal and Hydraulics Laboratory, Vicksburg, MS, U.S. Army Engineer Research and Development Center (ERDC), was the Lead Technical Director for Navigation Systems; Dr. Stanley C. Woodson, Geotechnical and Structures Laboratory (GSL), Vicksburg, MS, ERDC, was the INP Program Manager.

This report was prepared by Dr. Robert M. Ebeling, Information Technology Laboratory (ITL), Vickburg, MS, ERDC; Dr. Muluneh Azene, Southern University and A&M College, Baton Rouge, LA; and Mr. Ralph W. Strom, Consultant, Portland, OR. The research was monitored by Dr. Ebeling, Principal Investigator for WU 33272, under the supervision of Dr. Charles Welch, Chief, Engineering and Informatic Systems Division, ITL; Dr. Jeffery P. Holland, Director, ITL; and Dr. David R. Pittman, Acting Director, GSL.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL John W. Morris III, EN, was Commander and Executive Director.

1 Background Concepts, Procedures, and Guidelines Pertinent to the Design of Tall, Flexible Anchored Walls

Information pertinent to the design of tall, ***flexible*** (tieback) retaining wall systems constructed from the top down (e.g., Figure 7.3 in Strom and Ebeling 2001) and containing multiple rows of prestressed anchors in a homogeneous soil site is provided in the following paragraphs. Additional information relative to the design of tall, ***stiff*** tieback wall systems can be found in Strom and Ebeling (2002a).

1.1 Design of Flexible Tieback Wall Systems

Primarily because of its expediency in the practical design of tieback wall systems, the equivalent beam on rigid support method of analysis using apparent earth pressure envelopes is most often the design method of choice. This method provides the most reliable solution for flexible wall systems, i.e., soldier beam-lagging systems and sheet-pile wall systems, since for these types of systems a significant redistribution of earth pressures occurs behind the wall. Soil arching, stressing of ground anchors, construction-sequencing effects, and lagging flexibility all cause the earth pressures behind flexible walls to redistribute to and concentrate at anchor support locations (Federal Highway Administration (FHWA) FHWA-RD-98-066). This redistribution effect in flexible wall systems cannot be captured by equivalent beam on rigid support methods or by beam on inelastic foundation analysis methods where the active and passive limit states are defined in terms of Rankine or Coulomb coefficients.

Full-scale wall tests on flexible wall systems (FHWA-RD-98-066) indicated the active earth pressure used to define the minimum load associated with the soil springs behind the wall had to be reduced by 50 percent to match measured behavior. Since the apparent earth pressure diagrams used in equivalent beam on rigid supports analyses were developed from measured loads and thus include the effects of soil arching, stressing of ground anchors, construction-sequencing effects, and lagging flexibility, they provide a better indication of the strength performance of flexible tieback wall systems. This, however, is only applicable to those flexible wall systems in which

- Overexcavation to facilitate ground anchor installation does not occur.

- Ground anchor preloading is compatible with active limit state conditions.

- The water table is below the base of the wall.

The design of flexible wall systems with post-tensioned tieback anchors is illustrated in this report. The design of both flexible and stiff tall wall systems is discussed in Strom and Ebeling (2001). Stiff, tall wall design examples are provided in Strom and Ebeling (2002a).

1.1.1 Identifying flexible wall systems

Five Corps focus wall systems were identified in Strom and Ebeling (2001), as follows:

- Vertical sheet-pile system with wales and post-tensioned tieback anchors.

- Soldier beam system with wood or reinforced concrete lagging, and post-tensioned tieback anchors. For the wood lagging system, a permanent concrete facing system is required.

- Secant cylinder pile system with post-tensioned tieback anchors.

- Continuous reinforced concrete slurry wall system with post-tensioned tieback anchors.

- Discrete concrete slurry wall system (soldier beams with concrete lagging) with post-tensioned tieback anchors.

These systems are described in detail in Chapter 2 of Strom and Ebeling (2001).

Deformations and wall movements in excavations are a function of soil strength and wall stiffness, with wall stiffness a function of structural rigidity (EI) of the wall and the vertical spacing of anchors (L). Soil stiffness correlates to soil strength and, therefore, soil strength is often used in lieu of soil stiffness to characterize the influence of the soil on wall displacements. Steel sheet piles and steel soldier beams with timber lagging systems are considered to be flexible tieback wall systems. Secant cylinder pile, continuous concrete slurry wall, and discrete concrete slurry wall systems are considered to be stiff tieback wall systems.

The effect of wall stiffness on wall displacements and earth pressures is described in Xanthakos (1991) and in FHWA-RD-81-150. In this FHWA report it is indicated that, by finite element analyses, Clough and Tsui (1974) showed that wall and soil movements could be reduced by increasing wall rigidity and tieback stiffness. However, none of the reductions in movements were proportional to the increased stiffness. For example, an increase in wall rigidity of 32 times reduced the movements by a factor of 2. Likewise, an increase in the tieback stiffness by a factor of 10 caused a 50 percent reduction in movements.

Other investigators (FHWA-RO-75) also studied the effect of support stiffness for clays. They defined system stiffness by EI/L^4, where EI is the stiffness of the wall and L is the distance between supports (see Figure 1.1). The measure of wall stiffness is defined as a variation on the inverse of Rowe's flexibility number for walls, and is thus expressed by EI/L^4, where L is the vertical distance between two rows of anchors. Wall stiffness refers not only to the structural rigidity derived from the elastic modulus and the moment of inertia, but also to the vertical spacing of supports (in this case, anchors). It is suggested (in FHWA-RO-75, Figure 9-106) that, for stiff clays with a stability number ($\gamma H/s_u$) equal to or less than 3, a system stiffness (EI/L^4) of 10 or more would keep soil displacement equal to or less than 1 in.[1,2] However, other factors (prestress level, overexcavation, factors of safety, etc.) also influence displacement. Data in this figure clearly indicate that stiff wall systems in stiff clays will displace less than flexible wall systems in soft clays. Table 1.1 categorizes flexible and stiff wall systems with respect to the Corps focus wall systems of the Strom and Ebeling (2001) report.

Table 1.1
Stiffness Categorization of Focus Wall Systems

Focus Tieback Wall System Description	Wall Stiffness Category	
	Flexible	Stiff
Vertical sheet-pile system	✓	
Soldier beam system	✓	
Secant cylinder pile		✓
Continuous reinforced concrete slurry wall system		✓
Discrete concrete slurry wall system		✓

Using the approach in FHWA-RO-75, the wall stiffness can be quantified in terms of the flexural stiffness (EI) per foot run of wall and in terms of the relative flexural stiffness (EI/L^4). This information is presented in Table 1.2 for the focus wall systems of the Strom and Ebeling (2001) report. The relative flexural

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[1] At this time, the authors of this report recommend that, when tieback wall system displacements are the quantity of interest (i.e., stringent displacement control design), they be estimated by nonlinear finite element-soil structure interaction (NLFEM) analysis.
[2] A table of factors for converting non-SI units of measurement to SI units is presented on page ix.

stiffness in the table is based on a span length (L) (i.e., a vertical anchor spacing) of 10 ft.

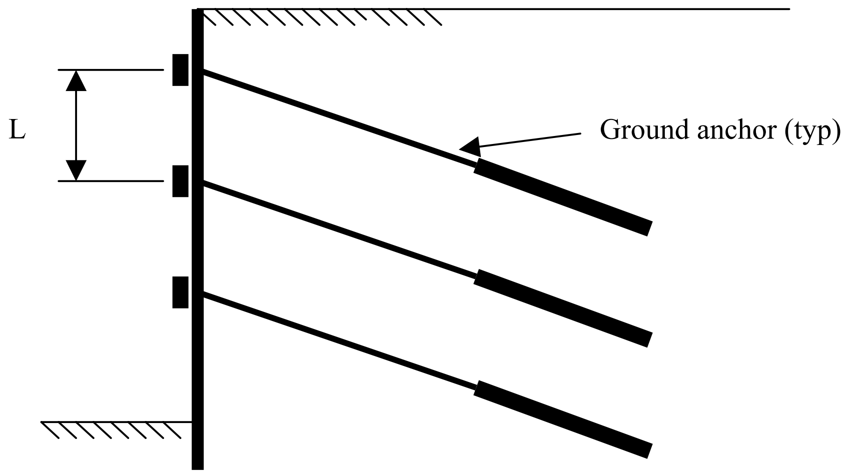

Figure 1.1. Definition of span length "L"

Table 1.2
General Stiffness Quantification for Focus Wall Systems

Wall Stiffness	Wall System	EI k-ft^2 / ft x 10^4	EI/L^4 ksf/ft
Flexible			
	Vertical sheet-pile system	0.3 to 5.0	3.7 (1)
	Soldier beam system	0.1 to 4.0	1.5 (2)
Stiff			
	Secant cylinder pile	8.0 to 250.0	239.8 (3)
	Continuous reinforced concrete slurry wall	30.0 to 150.0	123.1 (4)
	Discrete concrete slurry wall	35.0 to 160.0	92.3 (5)

(1) Relative stiffness based on PZ 27 sheetpiling, per Olmsted prototype wall.
(2) Relative stiffness based on HP12×53 soldier beams spaced at 8.0 ft on center (OC), per FHWA-RD-97-130 design example.
(3) Relative stiffness based on 5.0-ft-diam caisson piles spaced at 7.0 ft OC, per Monongahela River Locks and Dams 2 Project.
(4) Relative stiffness based on 3.0-ft-thick continuous slurry trench wall, per Bonneville Navigation Lock Temporary Tieback Wall.
(5) Relative stiffness based on W36×393 soldier beams spaced at 6.0 ft OC with concrete lagging, per Bonneville Navigation Lock upstream wall.

It should be recognized from the above stiffness calculations that a secant pile system with L equal to 28.5 ft would produce a flexural stiffness value of EI/L^4 equal to that for the vertical sheet-pile wall system with L equal to 10 ft. Therefore, it is possible by spacing anchors at close intervals to obtain a stiff wall system using flexible sheetpiling or, vice versa, to obtain a flexible wall system using secant piles with widely spaced anchors.

1.1.2 Tieback wall performance objectives

Depending on the performance objective, one of two design approaches can be used: "safety with economy" or "stringent displacement control" design procedures.

1.1.2.1 "Safety with economy" design. Common factors of safety used in practice for the design of anchored walls range between 1.1 and 1.5, applied to the shear strength of the soil and used in the calculation of the earth pressure coefficient that characterizes the magnitude of the total force applied to the wall (FHWA-RD-98-065). Values adopted for a factor of safety vary with the importance of the wall, the consequences of failure, the performance objective (i.e., safety with economy or stringent displacement control), and economics. Factors of safety ranging from 1.1 to 1.2 are generally considered unacceptable for the design of permanent walls. Walls constructed with factors of safety between 1.1 and 1.2 may be stable, but may also experience undesirable displacements near the wall (FHWA-RD-98-065). Therefore, factors of safety in this range should be used with caution and only for temporary walls where large displacements are considered to be acceptable.

The design and construction of a temporary excavation tieback wall support system with a low factor of safety (i.e., where large displacements were anticipated) is described in Cacoilo, Tamaro, and Edinger (1998). For permanent walls, in most situations some lateral movement of the tieback wall system can be tolerated, recognizing that, with lateral wall movement, settlements will occur in the retained soil immediately behind the wall. Tieback wall designs based on strength only, without special consideration of wall displacement, are termed "safety with economy" designs. For flexible wall systems, this means that the tieback anchors and wall system can be designed for soil pressure conditions "approaching" active state conditions (versus at-rest conditions). As such, the apparent earth pressure diagrams used in the design can be based on a total load approach using a factor of safety of 1.3 applied to the shear strength of the soil per the design recommendations of FHWA-RD-97-130. Trapezoidal earth pressure distributions are used for this type of analysis.

The general practice for "safety with economy" design is to keep anchor prestress loads to a minimum consistent with active, or near active, soil pressure conditions (depending upon the value assigned to the factor of safety). This means the anchor size would be smaller, the anchor spacing larger, and anchor prestress lower than that found in designs requiring stringent displacement control.

1.1.2.2 Stringent displacement control design. A performance objective for a tieback wall can be to restrict wall and soil movements during excavation to a tolerable level so that structures adjacent to the excavation will not experience distress. According to FHWA-RD-81-150, the tolerable ground surface settlement may be less than 0.5 in. if a settlement-sensitive structure is founded on the same soil used for supporting the anchors. Tieback walls designs that are required to meet specified displacement control performance objectives are termed "stringent displacement control" designs. Selection of the appropriate design pressure diagram for determining anchor prestress loading depends on the level of wall and soil movement that can be tolerated. Walls built with factors of safety between 1.3 and 1.5 applied to the shear strength of the soil may result in smaller displacements if stiff wall components are used (FHWA-RD-98-065). To minimize the outward movement, the design would proceed using soil pressures at a magnitude approaching at-rest pressure conditions, i.e., factor of safety of 1.5 applied to the shear strength of the soil.

It should be recognized that even though the use of a factor of safety equal to 1.5 is consistent with an at-rest (i.e., zero soil displacement condition) earth pressure coefficient (as shown in Engineer Manual 1110-2-2502, Figure 3-6) (Headquarters, Department of the Army 1989), several types of lateral wall movement could still occur. These include cantilever movements associated with installation of the first anchor; elastic elongation of the tendon anchor associated with a load increase; anchor yielding, creep, and load redistribution in the anchor bond zone; and mass movements behind the ground anchors (FHWA-SA-99-015).

It should also be recognized that a stiff rather than flexible wall system may be required to reduce bending displacements in the wall to levels consistent with the performance objectives established for the stringent displacement control design. However, a stringent displacement control design for a flexible wall system would result in anchor spacings that are closer and anchor prestress levels that are higher than those for a comparable safety with economy design. If displacement control is a critical performance objective for the project being designed, the use of a stiff rather than flexible wall system should be considered. (See Strom and Ebeling (2002a) for simplified design procedures for stiff tieback retaining walls.)

1.1.3 Progressive design of tieback wall systems

As with most designs, a progressive analysis starting with the simplest design tools and progressing to more comprehensive design tools when necessary is highly recommended by the authors. With respect to flexible wall systems, some of the more comprehensive analysis tools used for stiff wall system analysis (i.e., construction-sequencing analysis based on classical earth pressure distributions, and beam on inelastic foundation analysis) are not generally considered appropriate for the analysis of flexible wall systems. This is because apparent pressure diagrams, since they are "envelopes" based on measurements made during construction, include the effects of soil arching, wall flexibility, preloading of supports, facial stiffness, and construction sequencing. The most comprehensive design tool is a nonlinear finite element (NLFEM) soil-structure interaction (SSI) analysis. The NLFEM analysis is required when it becomes necessary to verify that the design meets stringent displacement control performance objectives. The design and analysis tools used in the design of flexible wall systems are summarized in Table 1.3 and described in the succeeding paragraphs.

Table 1.3
Design and Analysis Tools for Flexible Wall Systems

Analysis	Objective	Description	Analysis Method
RIGID 1	Final design when performance goal is "safety with economy." Preliminary design when performance goal is "stringent displacement control."	Beam on rigid supports analysis using apparent pressure "envelope" diagram. Apparent pressure diagram based on a total load approach. Total load is based on a factor of safety of 1.3 applied to the shear strength of the soil when the performance goal is "safety with economy." Total load is based on a factor of safety of 1.5 applied to the shear strength of the soil when the performance goal is "stringent displacement control."	Hand calculations
NLFEM	Final design when performance goal is "stringent displacement control."	Nonlinear soil-structure finite element construction-sequencing analysis.	PC SOILSTRUCT-ALPHA

1.2 RIGID 1 Method

In the RIGID 1 Method, a vertical strip of the tieback wall is treated as a multispan beam supported on rigid supports located at tieback points in the upper region of the wall. The lowermost rigid support is assumed to occur at finish grade. The wall is loaded on the driving side with an apparent pressure loading. In general practice, the use of soil pressure envelopes as loadings for a beam on rigid support analysis provides an expedient method for the initial layout, and sometimes the final design, of tieback wall systems. The soil pressure envelopes, or apparent earth pressure diagrams, however, were not intended to represent the real distribution of earth pressure, but instead constituted hypothetical pressures. These hypothetical pressures were a basis from which there could be calculated strut loads that might be approached but would not be exceeded during the entire construction process.

The apparent pressure loading used in the example problems is in accordance with FHWA RD-97-130. (See Figure 28 of this FHWA report for the apparent pressure diagram used for a wall supported by a single row of anchors and Figure 29 for the apparent pressure diagram used for a wall supported by multiple rows of anchors.) This information is also presented in Strom and Ebeling (2001, Figures 5.3 and 5.4). RIGID 1 design procedures are illustrated in the example problems contained in this report and in the example problems in Section 10 of FHWA-RD-97-130. When tiebacks are prestressed to levels nearer to active pressure conditions (versus at-rest conditions), the total load used to determine the apparent earth pressure is based on that approximately corresponding to a factor of safety of 1.3 on the shear strength of the soil. When tiebacks are prestressed to minimize wall displacements, the total load used to determine the apparent earth pressure is based on use of an at-rest earth pressure coefficient, or that approximately corresponding to a factor of safety of 1.5 applied to the shear strength of the soil. Empirical formulas are provided with the apparent pressure method for use in estimating anchor forces and wall bending moments.

1.3 NLFEM Method

When displacements are important with respect to project performance objectives, an NLFEM-SSI analysis should be performed. In an NLFEM analysis, soil material nonlinearities are considered. Displacements are often of interest when displacement control is required to prevent damage to structures and utilities adjacent to the excavation. To keep displacements within acceptable limits, it may be necessary to increase the level of prestressing beyond that required for basic strength performance. An increase in tieback prestressing is often accompanied by a reduction in tieback spacing. As tieback prestress is increased, wall lateral movements and ground surface settlements decrease. Associated with an increased level of prestress is an increase in soil pressures. The higher soil pressures increase demands on the structural components of the tieback wall system.

General-purpose NLFEM programs for two-dimensional plane strain analyses of SSI problems are available to assess displacement demands on tieback wall systems. These programs can calculate displacements and stresses due to incremental construction and/or load application and are capable of modeling nonlinear stress-strain material behavior. An accurate representation of the nonlinear stress-strain behavior of the soil, as well as proper simulation of the actual (incremental) construction process (i.e., excavation, anchor installation, anchor prestress, etc.) in the finite element model, is essential if this type of analysis is to provide meaningful results. See Strom and Ebeling (2001) for additional details regarding nonlinear SSI computer programs for displacement prediction.

1.4 Factors Affecting Analysis Methods and Results

1.4.1 Overexcavation

Overexcavation below ground anchor support locations is required to provide space for equipment used to install the ground anchors. It is imperative that the specified construction sequence and excavation methods are adhered to and that overexcavation below the elevation of each anchor is limited to a maximum of 2 ft. Construction inspection requirements in FHWA-SA-99-015 require inspectors to ensure that overexcavation below the elevation of each anchor is limited to 2 ft, or as defined in the specifications. Overexcavation exceeding 2 ft should be a “red flag” to the designer, indicating that a construction-sequencing evaluation is needed. A construction-sequencing analysis is likely to indicate that the maximum force demands on the wall and tiebacks will occur during intermediate stages of construction rather than for the final permanent loading condition. For additional information on the effect overexcavation has on tieback wall performance, see Yoo (2001).

1.4.2 Ground anchor preloading

Unless anchored walls are prestressed to specific active stress levels and their movement is consistent with the requirements of the active condition at each construction stage, the lateral earth pressure distribution will be essentially nonlinear with depth, and largely determined by the interaction of local factors. These may include soil type, degree of fixity or restraint at the top and bottom, wall stiffness, special loads, and construction procedures (Xanthakos 1991). To ensure that ground anchor prestress is consistent with active state conditions, the designer will generally limit anchor prestress to values that are between 70 and 80 percent of those determined using an equivalent beam on rigid supports analysis based on apparent pressure loadings (FHWA-RD-81-150). However, this may produce wall movements toward the excavation that are larger than tolerable, especially in cases where structures critical to settlement are founded adjacent to the excavation. Larger anchor prestressed loads are generally used when structures critical to settlement are founded adjacent to the excavation.

1.4.3 Construction long-term, construction short-term, and post-construction conditions

For a free-draining granular backfill, the pore-water pressure used in the analysis does not usually include excess pore-water pressures generated in the soil by changes in the total stress regime due to construction activities (excavation, etc.). This is because the rate of construction is much slower that the ability of a pervious and free-draining granular soil site to rapidly dissipate construction-induced excess pore-water pressures.

However, for sites containing soils of low permeability (soils that drain slower than the rate of excavation/construction), the total pore-water pressures will not have the time to reach a steady-state condition during the construction period. In these types of slow-draining, less permeable soils, the shear strength of the soil during wall construction is often characterized in terms of its undrained shear strength. These types of slow-draining, less permeable soils are often referred to as “cohesive soils.” The horizontal earth pressures are often computed using values of the undrained shear strength for these types of soils, especially during the short-term, construction loading condition (sometimes designated as the undrained loading condition, where the term undrained pertains to the state within the soil during this stage of loading).

As time progresses, however, walls retained in these types of soils can undergo two other stages of construction loading: the construction long-term (drained or partially drained) condition and the postconstruction/permanent (drained) condition. Under certain circumstances, earth pressures may be computed in poorly drained soils using the Mohr-Coulomb (effective stress-based) shear strength parameter values for the latter load case(s).

Liao and Neff (1990), along with others, point out that all three stages of loading must be considered when designing tieback wall systems, regardless of soil type. As stated previously, for granular soils, the construction short-term and long-term conditions are usually synonymous since drainage in these soils occurs rapidly. Differences in the construction short- and long-term conditions are generally significant only for cohesive soils. Changes in the groundwater level (if present) before and after anchor wall construction, as well as postconstruction/permanent, must also be considered in these evaluations. Designers must work closely with geotechnical engineers to develop a soils testing program that will produce soil strength parameters representative of each condition–the construction short term, construction long term, and postconstruction. The program should address both laboratory and field testing requirements. Additional information on construction short-term, construction long-term, and postconstruction condition earth pressure loadings can be found in Strom and Ebeling (2002a). Methods used to estimate long-term (drained) shear strength parameters for stiff clay sites are presented in Appendix A.

1.5 Types of Ground Anchors

1.5.1 General

The usual practice is for the wall designer to specify the anchor capacity and any right-of-way and easement constraints required of the anchorage system. It is up to the tieback anchor contractor, usually a specialty subcontractor, to propose the type of anchorage system to be used to meet the wall design requirements. Once an anchorage system is proposed, the tieback anchor contractor is generally required to conduct performance tests in the field to assure that the bond zone for the anchorage system selected is adequate to provide the desired capacity. (Refer to Strom and Ebeling (2002b) regarding performance testing of tieback anchors.) This section of the report provides an introduction to issues pertinent to anchor bond zone design. It is not intended as an all-encompassing reference on this subject, but is intended to provide background information with respect to the anchorage bond zone design procedures used in the example problems.

There are three main ground anchor types that are currently used in U.S. practice: (1) straight shaft gravity-grouted ground anchors (Type A); straight shaft pressure-grouted ground anchors (Type B); and (3) post-grouted ground anchors (Type C). Although not commonly used today in U. S. practice, another type of anchor is the underreamed anchor (Type D). These ground anchor types are illustrated in Figure 1.2 and are briefly described in the following sections.

Anchor bond lengths for gravity-grouted, pressure-grouted, and post-grouted soil anchors are typically 15 to 40 ft long (FHWA-SA-99-015, page 71). Significant increases in capacity for bond lengths greater than 40 ft cannot be

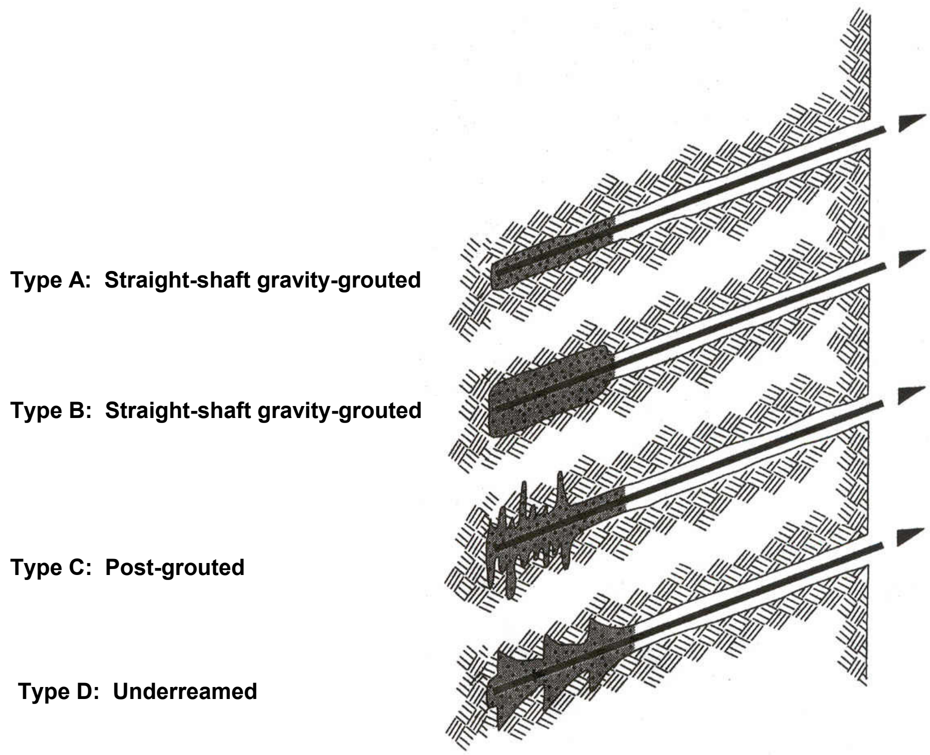

Figure 1.2. Main types of grouted ground anchors (after Figure 4, FHWA-SA-99-015)

achieved unless specialized methods are used to transfer load from the top of the anchor bond zone towards the end of the anchor.

Anchor design capacities are always verified by field testing. Should testing indicate anchor capacities to be insufficient, anchor capacities can increased by increasing the length of the bond zone or by increasing the diameter of the anchor (page 11 in Schnabel and Schnabel 2002). Alternatively, a higher capacity anchorage system may be substituted for the project or the use of more, lower capacity anchors can be suggested.

1.5.2 Straight-shaft gravity-grouted anchors

Straight-shaft gravity-grouted anchors are typically installed in rock and very stiff to hard cohesive soil deposits. Tremie (gravity displacement) methods are used to grout the anchor in a straight shaft borehole. The borehole may be cased or uncased depending on the stability of the borehole. Anchor resistance to pullout of the grouted anchor depends on the shear resistance that is mobilized at the grout/ground interface.

FHWA-RD-97-130 (page 25) observes that hole diameters vary between 3 and 16 in. for straight gravity-grouted anchors and that cased holes are normally 3 to 7 in. in diameter. For gravity grouting purposes, ground anchors are usually installed at inclinations between 15 and 30 deg down from horizontal (page 70 in FHWA-SA-99-015). Rotary, percussion, or combinations of both drilling methods, and hollow stem auger methods are used to advance the borehole. A casing may have to be used to maintain the borehole open in overburden or in fractured rock zones.

Load transfer for small-diameter gravity-grouted systems is generally estimated from empirical data since no theoretical relationship and corresponding material parameter values have been universally accepted that can accurately estimate their ultimate capacity. The drill hole diameter for these systems is generally equal to or less than 4 in. The ultimate capacity (TF_{ult}) of a small-diameter straight shaft, gravity-grouted anchor can be estimated by the following equation:

$$TF_{ult} = (L_b) \bullet (RLT_{ult}) \quad \text{(Equation 1.1)}$$

where

L_b = anchor bond zone length (feet)

RLT_{ult} = ultimate capacity of rate of load transfer (kips per foot)

Presumptive load transfer rates for small-diameter gravity-grouted anchor systems in soil can be found in Table 8.1, Strom and Ebeling (2001), or Table 6 of FHWA-SA-99-015. Presumptive load transfer rates for small-diameter gravity-grouted anchor systems in rock can be found in Table 8.3, Strom and Ebeling (2001), or Table 8 of FHWA-SA-99-015.

Load transfer for the larger diameter gravity-grouted anchor systems is a function of the shaft perimeter area. Alternatively, and as a first approximation, the ultimate capacity (TF_{ult}) of a large-diameter straight shaft, gravity-grouted anchor can be estimated as

$$TF_{ult} = (L_b) \bullet [\pi \bullet (d_a)] \bullet (\mathrm{ABS}_{\mathrm{ult}}) \quad \text{(Equation 1.2)}$$

where

d_a = diameter of the drill hole (feet)

ABS_{ult} = average ultimate soil-to-grout bond stress (kips per ft^2)

by adapting the equation given in Section 6.7.2.2 of Post-Tensioning Institute (PTI) (1996) (also given as Equation 4-16 in Xanthakos 1991). Section 6.7.2.2 of PTI (1996) notes that existing theoretical and empirical methods for predicting anchor capacity should only be used for preliminary design estimate purposes. The final (working load) anchor capacity shall be verified by proof testing each anchor in the field and performance testing select anchors (Strom and Ebeling 2002b).

Values for the average ultimate soil-to-grout bond stress for gravity-grouted anchors are provided for a variety of soils in Table 2.1 of Schnabel and Schnabel

(2002). Also, values for gravity-grouted anchors in soil and rock are given in Table 8.2 of Strom and Ebeling (2001) or Table 7 in FHWA-SA-99-015. Presumptive values for ABS_{ult} for hollow stem-augered tiebacks are given in Figure 24 of Andersen (1984) for a variety of soil types. (According to Andersen (1984), hollow-stem-augered tiebacks are constructed by first inserting the tieback tendon in the auger. A slip fit point is attached to the tendon and the point is inserted in the end of the auger. Next the tieback hole is drilled to the desired depth, and upon completion, grout is pumped down the auger as the auger is extracted.)

The value for the average ultimate soil-to-grout bond stress for cohesive soils has been related to the undrained shear strength S_u by

$$\mathrm{ABS}_{\mathrm{ult}} = \alpha \bullet S_u \qquad \text{(Equation 1.3)}$$

where α is a constant.

The value for α is sometimes referred to as the adhesion factor or as a reduction factor. Drilling techniques have a decisive effect on anchor capacity. In general, they tend to smear or soften the soil surface so that the soil-to-grout bond ultimate shear stress is often less than the shear strength of the soil. Frequently cited values for α range from 0.3 to 0.45 for gravity-grouted anchors (page 705 in Littlejohn 1990; page 171 in Xanthakos 1991). However, the range in the value for α can be larger. The α-value used will depend on the installation procedure used and whether the borehole is gravity-grouted or pressure-grouted (see following section). The cohesive soil example computation cited in Section 10.2 on page 203 of FHWA-RD-97-130 uses α equal to 0.725 for a 12-in.-diameter straight shaft gravity-grouted anchor created using a 12-in.-diameter auger with the cautionary note that this value was determined from load tests.

To improve the load-carrying capacity of bond anchor zones, other types of anchorage systems have been devised. They are discussed in the following paragraphs.

1.5.3 Straight-shaft pressure-grouted anchors

Straight-shaft pressure-grouted anchors are most suitable for coarse granular soils and weak fissured rock. This anchor type is also used in fine-grained cohesionless soils. With this type of anchor, grout is injected into the bond zone under pressures greater than 50 psi (0.35 MPa). The borehole is typically drilled using a hollow stem auger, or by rotary techniques with drill casings. Hollow-stem-augered systems are constructed by first inserting the tieback tendon in the auger. A slip fit point is attached to the tendon and the point is inserted in the end of the auger. Next, the tieback hole is drilled to the desired depth, and upon completion, grout is pumped down the auger as the auger is extracted. Pressure-grouted anchor systems can also be installed by driving and/or drilling a closed-end casing to the desired length. The tendon is then inserted into the casing. The casing is extracted a short distance using center hole hydraulic cylinders, and the closure point is driven free from the end of the casing. Grout is pumped down the casing while the casing is extracted. Grout pressures of 150 psi or more are maintained until the entire anchor bond length has been grouted. The net effect of this operation is to “heal” the damage done by drilling operations on what becomes the soil-to-grout load transfer zone. Pressure grouting increases resistance to pullout relative to gravity-grouting methods by (1) increasing the normal stress (i.e., confining pressure) on the grout bulb resulting from compaction of the surrounding material locally around the grout bulb; and (2) increasing the effective diameter of the grout bulb.

Schnabel and Schnabel (2002, page 4) note that a normal pressure-grouted tieback is about 3 in. in diameter with an anchor 15 ft long; a 3-in.-diameter pressure-injected anchor will have the capacity per lineal foot of a anchor three or more times the capacity of a larger 12-in. straight shaft gravity-grouted anchor.

Load transfer for small-diameter pressure-grouted systems is generally estimated from empirical data since no theoretical relationship has been developed to accurately estimate their ultimate capacity. The drill hole diameter for these systems is generally equal to or less than 4 in. (Andersen 1984).

The ultimate capacity (TF_{ult}) of small-diameter pressure-grouted anchors can be estimated using Equation 1.1. Presumptive load transfer rates for small-diameter pressure-grouted anchor systems are discussed in Andersen (1984). Figure 2.2 in Schnabel and Schnabel (2002) provides presumptive ultimate load-carrying capacity of small-diameter, pressure-grouted anchors as a function of bond zone length for a variety of cohesionless soils. Load transfer for the larger diameter anchor systems is a function of the shaft perimeter area, and therefore can be estimated using Equation 1.2. Note that the diameter of the hole is commonly used as the value for d_a in this equation for pressure-grouted anchors since the anticipated diameter of the anchor is difficult to estimate. Presumptive values for the average ultimate soil-to-grouted bond stress for large-diameter pressure-grouted anchor systems are given for a variety of soils in Table 8.2 of Strom and Ebeling (2001), or Table 7 in FHWA-SA-99-015. Section 6.7.2.3(B) in PTI (1996) observes that pressure-grouted anchors in cohesionless soil develop capacities far in excess of the load expected from applying conventional soil mechanics theory. Section 4-8 in Xanthakos (1991) discusses failure of anchors in sand.

Designers may be required to use a post-grouted (regroutable) anchor system when sufficient capacity cannot be obtained using standard pressure-grouting methods.

1.5.4 Post-grouted (regroutable) ground anchors

Post-grouted ground anchors use delayed multiple grout injections to enlarge the grout body of the gravity-grouted ground anchors. Each injection is separated by one or two days. Post-grouting is accomplished through a sealed grout tube installed with the tendons. The tube is equipped with check valves in the bond zone. The check valves allow additional grout to be injected under high pressure into the initial grout, which has set. The high-pressure grout fractures the initial grout and wedges it outward into the soil, enlarging the grout body. Two fundamental types of post-grout anchors are used. One system uses a packer to isolate each valve. The other system pumps the grout down the post-grout tube without controlling which valves are open. Post-grouting was first tried in West Germany.

Grout pressures as high as 300 psi are used. The mechanism by which a regroutable anchor develops its capacity is not well understood. Available data show that post-grouting improves the capacity of tiebacks in cohesive soils. In most granular and cohesive soils it is possible to increase the anchor capacity by regrouting (page 4 in Schnabel and Schnabel 2002). Depending on the soil, the type of post-grouting system used, and the number of regrouts, anchor capacity increases ranging from 25 percent to more than 300 percent are possible (Andersen 1984). Schnabel and Schnabel (2002, page 4) note that in granular soils it is possible to increase the anchor capacity beyond the shear strength of the soil due to the induced radial stresses within the soil around the bond zone. Schnabel and Schnabel go on to observe that in overconsolidated clays, the regrouting tends to increase the shear imparted by the clay on the anchor and closer to the shear strength of the overconsolidated clay. Littlejohn (1990, page 702) observes that while this anchorage type is commonly applied in fine cohesionless soils, success has also been achieved in stiff cohesive deposits.

Littlejohn (1990, page 705) provides the following observations. Based on full-scale tests, theoretical skin frictions[1] for borehole diameters of 3 to 6 in. are known to increase with increasing consistency and decreasing plasticity. In stiff clays ($[l_c]^2$ = 0.8 to 1.0) with medium to high plasticity, skin frictions may be as low as 4.4 to 11.3 psi, while the highest values (of greater than 58 psi) are obtained in sandy silts of medium plasticity and very stiff to hard consistency (l_c = 1.25). The technique of post-grouting is also known to generally increase the skin friction of very stiff clays by some 25 to 50 percent according to Littlejohn, but better results are claimed in stiff clays of medium to high plasticity according to Xanthakos (1991, page 182).

Xanthakos (1991, page 181) notes that reported successful applications show increase in shear resistance along the interface from 17.4 psi to nearly 43.5 psi for stiff clay of medium to high plasticity, an increase of 150 percent. Data contained in Figure 4-31 of Xanthakos (1991) shows the theoretical skin friction (i.e., shear bond) increases with increasing grouting pressures (up to a limiting pressure of about 450 psi) for boreholes from 3 to 6 in. in diameter for clays of medium to high plasticity.

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[1] The theoretical skin friction is calculated using the ultimate load holding capacity, the borehole diameter, and the designed length of the bond zone for the anchor.

[2] The consistency index, $l_c = \dfrac{L_L - wc}{L_L - P_L}$

where L_L is the liquid limit, P_L is the plastic limit, and wc is the water content, all expressed in percent.

Data contained in Figure 4-32 of Xanthakos (1991) show a quantitative example of the increase in ultimate anchor capacity with each of two subsequent regrouting stages in a 4.5- to 4.8-in.-diameter borehole, for a total of three grouting stages (staged grout pressures of 70-130 psi, 215-230 psi, and 400-425 psi) in a gypsum-bearing marl formation. The in situ undrained shear strength of marls ranges from 0.6 to 1.6 ksf. For the same (20-ft) fixed length the ultimate anchor capacity is almost three times larger than the ultimate load at first grouting.

Littlejohn (1990, page 705) notes that Type C (Figure 1.2) anchorage design is based on the assumption of uniform skin friction, and safe working loads of 67 to 112 kips are common.

Cacoilo, Tamaro, and Edinger (1998) describe an application of post-grouted ground anchors in "soft" clay for a temporary tieback wall in regions that were not adjacent to operating warehouse buildings. The bond zone for the tiebacks was located in a deposit of marine clay and silt, commonly known as Boston Blue Clay; this marine clay and silt is described as overconsolidated, with the upper part of the stratum being highly desiccated and very stiff to hard (S_u ranges from approximately 1,250 to 2,300 psf). Below this desiccated crust, there is a zone of stiff to medium stiff clay, then a zone of sensitive, soft to medium stiff clay (S_u ranges from approximately 1,150 to 1,700 psf). For the tieback system to be feasible, a minimum anchor working capacity of 173 kips (with a minimum factor of safety of two) was required and was achieved through the use of special drilling procedures and post-grouting in the anchor bond zone. The target maximum test load for the anchors was established at 409 kips. A 40-ft-long bond anchor zone was established for the two-tier system through the temporary sheet piles, with a 5-ft anchor spacing and a 30-degree angle from horizontal. The upper tier was anchored in the desiccated clay crust, and the lower tier anchors were anchored in the softer clay below the crust. The production tiebacks were typically drilled by advancing a 7.5-in. outside diameter steel casing (through fill) to the bottom of the hole using internal flush, rotary drilling methods. The casing was cleaned out with the roller bit, and the cuttings were typically flushed with water. After the casing was flushed with water, the drill string was withdrawn and the casing was filled with grout placed by tremie methods. The tendon assembly (eight 7-wire strands) was then inserted into the grout-filled casing and the casing withdrawn. Each production tieback was post-grouted (using a mechanical type packer lowered into the valve and pumping in cement grout) four to five times.

1.5.5 Underreamed anchors

To improve on the capacity of straight-shaft pressure-grouted anchors, either post-grouted or underreamed anchors can be used. This section provides a method for estimating the capacity of underreamed anchors. Littlejohn (1990) cites a case where a 6-in.-diameter augered hole with a straight shaft gravity-grouted anchor with a 35-ft bond length failed at 225 kips, whereas an underreamed anchorage with a bond length of 10 ft withstood a load of 337 kips without any sign of failure. Underreamed anchors consist of tremie grouted boreholes that include a series of enlargement bells or underreams. This type of anchor may be used in firm to hard cohesive deposits. Underreamed anchors can be used in stiff, overconsolidated clays when the undrained shear strength exceeds 1,900 psf (Littlejohn 1990, page 706). Refer to Littlejohn (1990) and Xanthakos (1991) for additional details regarding this type of anchorage. In addition to resistance through side shear, as is the principal load transfer mechanism for other anchors, resistance may also be mobilized through end bearing. Care must be taken to form and clean the underreams.

As a first approximation and using the formulation given in Section 4-9 of Xanthakos (1991), the ultimate capacity (TF_{ult}) of underreamed anchors in stiff, overconsolidated clays can be estimated as

$$TF_{ult} = T_{underream} + T_{endbearing} + T_{shaft} \qquad \text{(Equation 1.4)}$$

where

$T_{underream}$ = side shear in underream length

$T_{endbearing}$ = end bearing in clay

T_{shaft} = side shear along shaft length

The three ultimate force components that contribute to the ultimate capacity of the underreamed anchor are computed using:

$$T_{underream} = \pi \bullet D \bullet L_u \bullet (S_u \bullet a_u) \qquad \text{(Equation 1.5)}$$

$$T_{endbearing} = \frac{\pi}{4} \bullet (D^2 + d^2) \bullet N_c \bullet S_u \qquad \text{(Equation 1.6)}$$

$$T_{shaft} = \pi \bullet d \bullet L_s \bullet (S_u \bullet a_s) \qquad \text{(Equation 1.7)}$$

where

D = diameter of underream

L_u = length of the underream section

S_u = average undrained shear strength of the stiff clay

a_u = efficiency coefficient, usually in the range 0.75-0.95 (according to Xanthakos 1991), reflecting soil disturbance

d = diameter of shaft

N_c = bearing capacity factor, which ranges from 6 to 13 for stiff to hard clays; an N_c value close to 9 is often used (according to Xanthakos 1991)

L_s = shaft length (part of fixed length)

a_s = shaft adhesion factor

On page 176 Xanthakos (1991) theoretically shows that for optimum design, underream spacing should be less than three times the underream diameter. Xanthakos gives the typical range for underream diameter D as 14-16 in. and typical values for shaft diameter d as 5-6 in. At this time underreamed anchors are not commonly used in the United States.

1.5.6 Rock anchors

Rock anchor systems are constructed by drilling a 3- to 6-in. diameter hole into rock. Rotary, percussion, or combinations of both drilling methods are used to advance the borehole. A casing may have to be used to maintain the borehole open in overburden or in fractured rock zones. After the hole has been drilled, a grout tube and tendon are inserted and grout pumped down the grout tube until the anchorage bond length has been completely grouted. Rock anchor system tiebacks are also shaft tiebacks, and as such their ultimate capacity can be estimated using the same equation provided for large-diameter pressure-injected anchor systems (Equation 1.2). Presumptive load transfer rates (ABS_{ult}) for rock anchor systems can be found in Table 8.2 of Strom and Ebeling (2001) or Table 7 in FHWA-SA-99-015 for a variety of rock types. Additionally, presumptive values for ABS_{ult} for permanent rock tiebacks are given in Figure 25 of Andersen (1984).

1.6 Example Problems

Design examples include soldier beam with timber lagging and sheet piles with wales and post-tensioned tieback anchored wall systems with multiple rows of tieback anchors. Wall heights of 25, 35, and 50 ft are considered, all with a horizontal retained soil surface.

Chapter 2 of this report deals with the application of procedures and guidelines described previously to soldier beam with timber lagging systems and cohesionless soil backfill. Detailed design examples using “safety with economy” and “stringent displacement control” performance objectives are provided for 50-ft wall heights. In Chapter 3, design procedures for cohesive soil backfill conditions are given.

Design examples for 50-ft sheet piles with wale systems retaining cohesive soil are presented in Chapter 4. The “safety with economy” and “stringent displacement control” design procedures are employed.

Summaries of results for 25- and 35-ft soldier beam and sheet-pile walls using each of these approaches are also provided for comparison in Chapter 5.

Theoretically, the number of rows of tieback anchors is computed to satisfy strength requirements and deformation constraints. However, site considerations and the risks associated with failure of one or two anchors in a single-tier anchor system suggest the application of more than one row of anchors for wall heights greater than 20 ft. Accordingly, minimum numbers of rows of anchors assumed are two, three, and four, respectively, for 25-, 35-, and 50-ft-high walls.

1.7 A Note of Caution Regarding Ground Anchors in Cohesive Soils

Design examples for tieback walls in a stiff, overconsolidated clay for a homogeneous soil site are given in Chapters 3 through 5 of this report. A stiff clay site was selected because soft to medium clay soils with stability numbers ($\gamma H/S_u$) greater than 5 are considered to be potentially dangerous and, as such, the use of a soldier beam and lagging system for support is questionable (see Table 12 of FHWA-SA-99-015). The Chapter 3 design computations for a soldier beam and lagging tieback wall system assume a stiff cohesive soil site with soil properties identical to those of the "cohesive soil" used in the design example given in Section 10.2 in FHWA-RD-97-130. Figure 106 of FHWA-RD-97-130 describes this "cohesive soil" as a silty clay with lenses and layers of fine sand (CL), stiff to hard. On page 204 of this FHWA report the undrained shear strength S_u is given as 2,400 psf, a unit weight of 132 pcf, an overconsolidation ratio (OCR) of 3, and a plasticity index of 19. This same cohesive soil is assumed for all subsequent tieback wall design examples in a homogeneous cohesive soil.

Tieback walls retaining stiff cohesive soils (for the undrained condition) are to be designed using nonsymmetrical apparent earth pressure diagrams identical in shape to the ones recommended for granular soils in Strom and Ebeling (2001). This would be Figure 5.3 in Strom and Ebeling (2001) for walls supported by one row of anchors (Figure 28, FHWA-RD-97-130), and Figure 5.4 in Strom and Ebeling (2001) for walls supported by multiple rows of anchors (Figure 29, FHWA-RD-97-130).

Provided there is no potential for a deep-seated failure, tieback walls retaining soft to medium clays (temporary support use only - undrained condition) are to be designed using the apparent earth pressure diagram of Figure 5.6 Strom and Ebeling, 2001 (Figure 32, FHWA-RD-97-130). The total earth pressure load for tieback walls in soft to medium clays with a deep-seated failure potential must be determined by limiting equilibrium methods using general-purpose slope stability program (GPSSP) analysis techniques.

The transition from using a stiff clay apparent earth pressure diagram to a soft to medium clay diagram does not occur at a unique undrained shear strength. For a given wall height or excavation depth, H, the undrained strength of the soil must satisfy Equation 1.8 in order to use the stiff clay apparent earth pressure diagrams (FHWA-RD-97-130, page 65).

$$S_u \geq \frac{H}{4}\left(\gamma - 22.857\right) \qquad \text{(Equation 1.8)}$$

where the units of H are in feet, total unit weight γ in pcf, and S_u in psf.

FHWA-SA-99-015, page 30, cautions tieback wall designers in cohesive soils regarding the creep of anchors in cohesive soils, specifically with regard to the failure of the ground-grout bond. Failure at the ground-grout interface may be characterized by excessive deformations under sustained loading, i.e., creep. Soil deposits that are potentially susceptible to excessive creep deformations include (1) organic soils; (2) clay coils with average liquidity index, LI, greater than 0.2; (3) clay soils with an average liquid limit, LL, greater than 50; and (4) clay soils with an average plasticity index, PI, greater than 20. Conservative anchor design loads and working bond stress values are recommended by this FHWA report for design involving permanent anchor installations in such soils, unless based on results from a predesign or preproduction test program.

The liquidity index can be used as an indication of overconsolidation in a “cohesive soil”: a low LI value indicates that the moisture content for the soil is relatively close to the PL of the soil, which indicates a potentially overconsolidated soil. A LI value close to 1.0 indicates that the moisture content is relatively close to the LL for the soil, which indicates a potentially normally consolidated or soft soil.

The extended creep test is used to evaluate creep deformations of anchors. An extended creep test is a long-duration test (e.g., approximately 8 hours), as discussed in Strom and Ebeling (2002b). Section 7.4.4 in FHWA-SA-99-015 states that these tests are required in a cohesive soil having a plasticity index greater than 20 or a liquid limit greater than 50. This FHWA report notes that for these ground conditions, a minimum of two ground anchors should be subjected to extended creep testing. Where performance or proof tests require extended load holds, extended creep tests should be performed on several production anchors. Schnabel and Schnabel (2002, page 47) observe that they are not aware of any instance in which a tieback, anchored in soft soil, and carefully tested in accordance with PTI recommendations, has failed in use.

FHWA-RD-97-130 notes on page 24 that anchors are routinely installed in soft rocks, clays, tills, and mixed soils and that recently, post-grouted anchors in clays have been used to support permanent retaining walls. The FHWA report states that permanent ground anchors are not normally installed in soils with high organic content, normally consolidated clays, and cohesive soils with an unconfined compressive strength less than 1 tsf. Anchors installed in soils with a liquidity index less than 0.2 perform satisfactorily. Successful permanent anchor installations have been built in soils with liquidity indices greater than 0.2. Lastly, in low-strength clays or soils with high liquidity indices, local experience or a precontract test program is recommended by this FHWA report.

1.8 Research and Development Needs

The FHWA-based design methodologies described herein with respect to flexible tieback wall systems assume that wall movements are consistent with the apparent earth pressures assumed for design. Lateral earth pressures, however, will be essentially nonlinear and dependent on many factors, including soil type, wall fixity and restraint, factors of safety, tieback size and spacing, tieback prestress levels, construction sequencing, overexcavation at anchor locations, and wall performance requirements. It is well known that it is impossible to predict wall displacements using a RIGID 1-type design procedure.

Additional research using nonlinear SSI finite element analyses is needed to provide insight into displacements for walls resulting from the use of the FHWA-based design and analysis tools illustrated in this report and the example problems. The research should be directed toward validating the simple design procedures used herein as suitable tools for designing anchors and for estimating wall moment demands on Corps project. In addition, the research should determine if there are simple analysis procedures that can be used to predict the displacement response for those Corps of Engineers tieback walls that must meet stringent displacement performance objectives.

2 Simplified Design Procedures for 50-ft-High Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Retaining Granular Soil

The two example problems presented in this chapter deal with the application of the design procedures and guidelines for drilled-in soldier beam systems with wood lagging as outlined in Strom and Ebeling (2001), FHWA-RD-97-130, and FHWA-SA-99-015. A 50-ft wall height with granular retained soil (horizontal retained soil surface), a homogenous loose sand, is considered. These design computations follow the granular soil design example of Section 10.1 in FHWA-RD-97-130 for the drilled-in soldier beam wall (starting on page 188). A "safety with economy" design example is given first, followed by a "stringent displacement control" design example. The soil properties used are in accordance with the granular soil, from the example given in Section 10.1 of FHWA-RD-97-130 (page 171):

- Friction angle, ϕ = 30 deg
- Unit weight, γ = 115 pcf
- Uncorrected Standard Penetration Test (SPT) resistance = 15 blows per ft

2.1 "Safety with Economy" Design

The earth pressure factor (EPF) for a typical coarse-grained soil can be obtained from Table 8 of FHWA-RD-97-130 or Table 5.3 of Strom and Ebeling (2001). For the soil properties described in the preceding paragraph, the EPF is 22.97 psf. The total earth pressure load (P_{tl}) used to develop an apparent earth pressure diagram is the equal to the earth pressure factor times the square of the wall height (H), or P = EPF $(H)^2$.

FHWA investigators (FHWA-RD-98-065) demonstrated that the total loads from Terzaghi, Peck, and Mesri's (1996) sand and soft to medium clay apparent pressure diagrams are equal to the total lateral loads using limiting equilibrium analyses with a factor of safety of about 1.3 applied to the shear strength of the soil. For the Corps' "safety with economy" design, a limiting equilibrium approach will be used with a factor of safety of 1.3 applied to the shear strength of the soil. (The factor of safety for the limiting equilibrium analysis is increased to 1.5 when a stringent displacement control design is required.) The total earth pressure load (P_{tl}) is determined based on the limiting equilibrium analysis. Limiting equilibrium calculations for the "safety with economy" design are provided in the following paragraphs. This process produces an EPF equal to 24.3 pcf, compared with 22.97 pcf obtained from the tables mentioned in the preceding paragraph. These tables use a tabulation from FHWA-RD-97-130 that includes a factor of safety comparable to that of the Corps' "safety with economy" design.

The total earth pressure load is determined by assuming that the shear strength of the soil is factored by the target factor of safety FS such that

$$\phi_{mob} = \tan^{-1}(\tan\phi / FS)$$

2.1.1 Effective pressure

The following calculations demonstrate the use of limiting equilibrium methods to determine the total earth pressure load (P_{tl}), or the external force required for stability of the tieback wall system. The effective pressure (p_e) for the FHWA nonsymmetrical earth pressure diagram (Figure 29 of FHWA-RD-97-130 or Figure 5.4 of Strom and Ebeling 2001) can then be determined from the total earth pressure load.

$$\phi_{mob} = \tan^{-1}\left(\frac{\tan\phi}{1.3}\right)$$

$$= \tan^{-1}\left(\frac{\tan 30°}{1.3}\right) = 23.95°$$

$$K_A = \tan^2\left(45° - \frac{\phi_{mob}}{2}\right) = 0.423$$

$$P_{tl} = K_A \gamma \frac{H^2}{2} = 0.423 * 115 * \frac{50^2}{2} = 60806 \text{ lb/ft}$$

$$\text{Effective pressure factor, EPF} = \frac{P_{tl}}{H^2} = \frac{60806}{50^2} = 24.3 \text{ pcf}$$

2.1.2 Apparent earth pressure diagram

The apparent earth pressure diagram and formulas for a tieback wall supported by multiple rows of anchors is as shown in Figure 5.4 of Strom and Ebeling (2001) and is illustrated for this particular example in Figure 2.1.

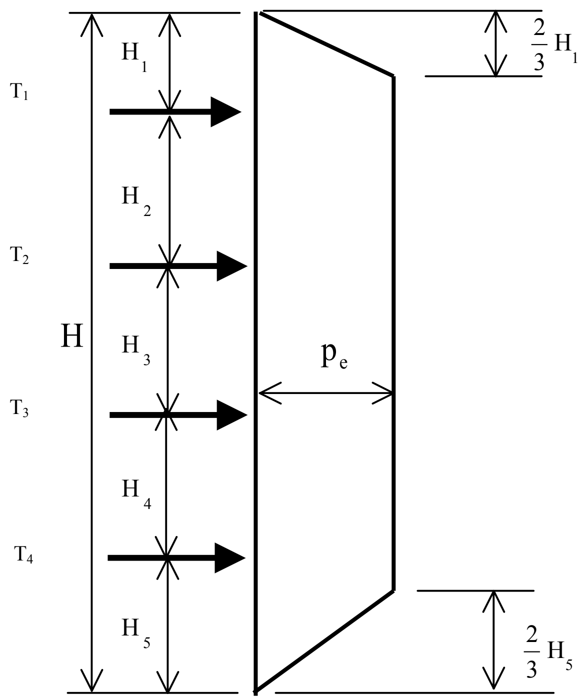


Figure 2.1. Apparent earth pressure

2.1.3 Anchor points

Using the empirical apparent earth pressure of Figure 2.1 and assuming four-tier anchoring, the vertical anchor spacing for balanced moments (i.e., upper cantilever moment, M_1, equal to maximum lower continuous span moment, MM_1) is determined as follows for the "safety with economy" design:

Setting $MM_1 = M_1$ $\qquad\qquad (_{i=1,2,3}\ldots)$

$$\frac{1}{10} H^2_{(2,3,4,5)}\ p = \frac{13}{54} H_1^2\, p$$

where

$H_{(2,3,4,5)}$ denotes vertical distances between anchors, i.e., H_2, H_3, H_4, H_5, assumed equal i.e.,

$$H_{(2,3,4,5)} = \sqrt{\frac{130}{54}} H_1 = 1.55 H_1$$

thus, with $H_2 = H_3 = H_4 = H_5$

$$H = H_1 + H_2 + H_3 + H_4 + H_5 = H_1 + 4\,(1.55 H_1)$$
$$50 = 7.2 H_1$$

therefore,

$$H_1 \approx 6.944 \text{ ft}$$

and,

$$H_2 = H_3 = H_4 = H_5 = \frac{H - H_1}{\#\text{ of vertical anchor spacing}} = \frac{50 - 6.944}{4}$$
$$= 10.764 \text{ ft}$$

Try H_1 = 7'-0" and $H_2 = H_3 = H_4 = H_5$ = 10' - 9"

Using these anchor spacings (in the vertical direction), the effective earth pressure, p_e, for the FHWA nonsymmetrical apparent pressure diagram can be determined:

$$P_e = \frac{P_{tl}}{(H - \frac{H_1}{3} - \frac{H_5}{3})} \qquad \text{(see Figure 5.4b, Strom and Ebeling 2001)}$$

$$= \frac{60806}{50 - \frac{7}{3} - \frac{30.75}{3}} = 1379 \text{ psf}$$

2.1.4 Bending moments on soldier beam

Using the information contained in Figure 5.4b of Strom and Ebeling (2001), the cantilever moment (M_1) and maximum interior span moments (MM_1) can be determined. (Moments are per foot of wall.)

$$M_1 = \frac{13}{54} H_1^2 p = \frac{13}{54} * 7.0^2 * 1379 = 16267 \text{ lb} - \text{ft/ft}$$

and,

$$MM_{1,2,3} = \frac{1}{10}(\text{larger of } H_{2,3,4,5})^2 \; p$$
$$= \frac{1}{10}(10.75)^2 * (1379)$$
$$= 15936 \text{ lb} - \text{ft/ft}$$

hence,

Maximum moment M_{max} = 16267 lb - ft /ft (also spacing OK for balanced moments)

2.1.5 Ground anchor load horizontal components

Also, using the information contained in Figure 5.4b of Strom and Ebeling (2001), the horizontal components of each anchor load, on a per foot run of wall basis, are determined. Assume soldier spacing (s) = 6 ft.

Top tier:

$$T_1 = \left(\frac{2}{3}H_1 + \frac{1}{2}H_2\right)p$$
$$= \left(\frac{2}{3} * 7.0 + \frac{1}{2} * 10.75\right)1379$$
$$= 13847 \text{ lb/ft}$$

(Design anchor force = 13.847 kips/ft × 6 ft/cos 20° = 88.4 kips)

Second tier:

$$T_2 = \frac{1}{2}\left(H_2 + H_3\right)p$$
$$= \frac{1}{2}\left(10.75 + 10.75\right)1379$$
$$= 14824 \text{ lb/ft}$$

(Design anchor force = 14.824 kips/ft × 6 ft/cos 20° = 94.7 kips)

Third tier:

$$
\begin{aligned}
T_3 &= \frac{1}{2}\left(H_3 + H_4\right)p \\
&= \frac{1}{2}\left(10.75 + 10.75\right)1379 \\
&= 14824 \text{ lb/ft}
\end{aligned}
$$

(Design anchor force = 14.824 kips/ft × 6 ft/cos 20° = 94.7 kips)

Lower tier:

$$
\begin{aligned}
T_4 &= \left(\frac{H_4}{2} + \frac{23}{48}H_5\right)p \\
&= \left(\frac{10.75}{2} + \frac{23 * 10.75}{48}\right)1379 \\
&= 14515 \text{ lb/ft}
\end{aligned}
$$

(Design anchor force = 14.515 kips/ft × 6 ft/ cos 15° = 90.2 kips)

T_{max} = 14824 lb/ft (Spacing OK for approximately balanced T)

2.1.6 Subgrade reaction using tributary method

Again, using the information contained in Figure 5.4b of Strom and Ebeling (2001), the subgrade reaction (R) is determined. As with the other quantities, the subgrade reaction is per foot run of wall.

$$
R = \left(\frac{3}{16}H_{n+1}\right)p
$$

i.e.,

$$
\begin{aligned}
R &= \frac{3}{16} * 10.75 * 1379 \\
&= 2780 \text{ lb/ft}
\end{aligned}
$$

2.1.7 Soldier beam size

Assume soldier beam spacing (s) = 6.0 ft.

Note the minimum permissible spacing is 4.0 ft (see Figure 8.5 of Strom and Ebeling 2001).

Hence, the maximum soldier beam design moment (M_{max}) is

$$M_{max} = \frac{16267}{1000} * 6 \ = \ \text{ft-lb} \ = \ 97.6 \, \text{ft-kip}$$

In accordance with Corps criteria (Headquarters, U.S. Army Corps of Engineers (HQUSACE) 1991), the allowable stresses for the soldier beams and wales shall be as follows:

Bending (i.e., combined bending and axial load): $f_b = 0.5\, f_y$

Shear $f_v = 0.33\, f_y$

Allowable stresses are based on 5/6 of the American Institute of Steel Construction Allowable Stress Design (AISC-ASD) recommended values and reflect the Corps' design requirements for steel structures. Thus,

The allowable bending stress for Grade 50 steel: $F_b = 0.5\, F_y = 25$ ksi

Allowable shear stress for Grade 50 steel: $F_v = 0.33\, F_y = 16.5$ ksi

The required soldier beam section modulus (S) for Grade 50 steel is, therefore,

$$S = \frac{M_{max}}{F_b} = \frac{97.6 * 12}{25} = 46.8 \text{ in.}^3$$

From AISC (1989), HP 10×57 provides $S_{xx} = 58.8 > 46.8$ in.^{3} OK

or

2 MC 10×28.5 provides $S_{xx} = 50.6 > 46.8$ in.^{3} OK

Try 2MC 10×28.5 Grade 50 steel for the "safety with economy" design.

Check shear capacity:

Maximum shear force, $V_{max} = T_{max} * 6 = 14824*6 = 88944$ lb =88.9 kips

Required area, $A = 88.9/16.5 = 5.39$ in.^{2}

Shear area provided by 2 MC 10×28.5,

$= 2*d*t_w = 2*10*0.425 = 8.5$ in.^{2} > 5.39 in.^{2} OK

where d and t_w are web depth and width for MC 10×28.5.

Use 2 MC 10 × 28.5 Grade 50 sections.

2.1.8 Anchor lengths

For constructibility, the upper three tiers of ground anchors will be inclined downward at an angle of 20 deg, and the lower tier inclined downward at an angle of 15 deg (see Figure 2.2). Using the unbonded length requirements of Figure 8.5 of Strom and Ebeling (2001), the minimum unbonded length for each anchor tier can be determined. These calculations are provided in the following subsection (2.1.8.1).

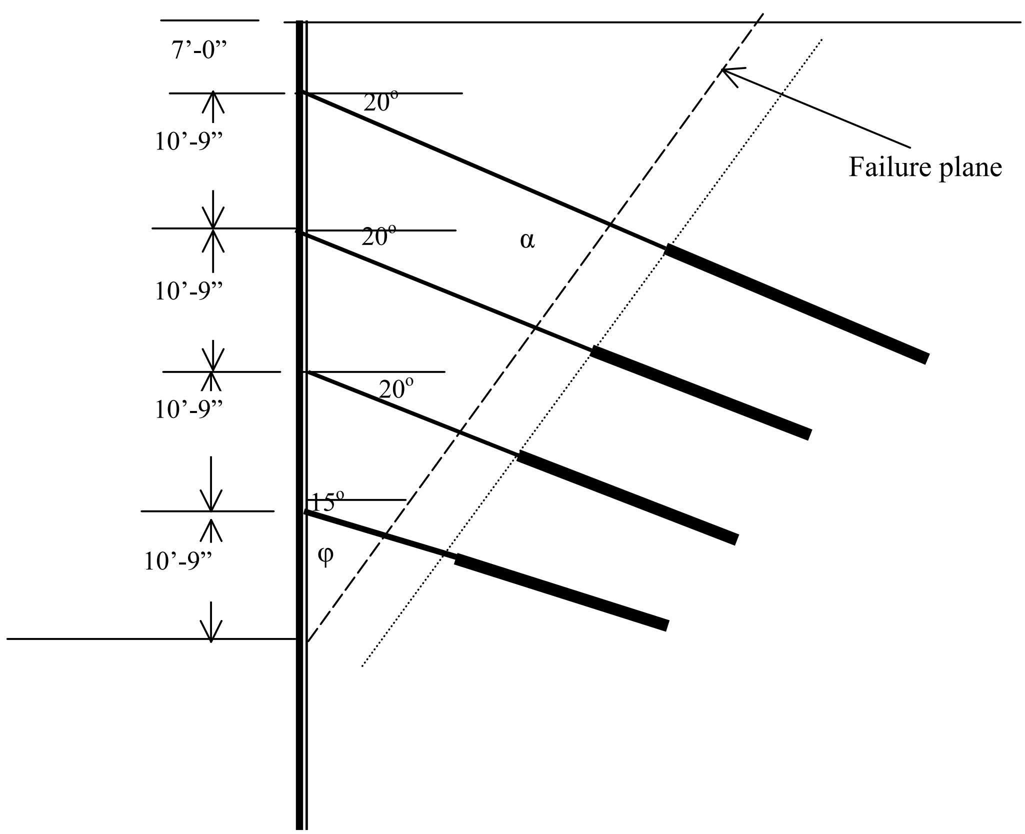


Figure 2.2. Anchors and placement

2.1.8.1 Unbonded anchor length, L_i. Assume 20-deg inclination for top three anchors and 15 deg for bottom tier anchor.

$$\frac{\text{unbonded length, L}}{\varphi} = \frac{\text{height of anchor point}}{\alpha}$$

$$\varphi = 45° - \frac{\phi}{2} = 45° - 15° = 30°$$

$$\alpha = 180° - 70° - 30° = 80°$$

Top-tier anchor:

$$\frac{L}{\sin 30} = \frac{43.00}{\sin 80}$$

$$L = \frac{43.00 * \sin 30}{\sin 80} = 21.8 \text{ ft}$$

Allowing 5 ft or 0.2H clearance beyond shear plane (see Figure 8.5, Strom and Ebeling 2001)

$$L_1 = 21.8 + (0.2\text{H or 5 ft, whichever is greater})$$

$$= 21.8 + 10 \ = 31.8 \text{ ft} > 15 \text{ ft minimum required for strand anchor} \quad \text{OK}$$

$$(\text{Minimum required for bar anchor is } 10 \text{ ft})$$

Similarly,

Second-tier anchor:

$$L_2 = \frac{32.25}{43} * L + 0.2H$$

$$= 26.35 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

Third-tier anchor:

$$L_3 = \frac{21.5}{43} * L + 0.2H$$

$$= 20.9 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

Lower-tier anchor:

$$L_4 = \frac{10.75}{\sin 75} * \sin 30 + 0.2H$$

$$= 15.56 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

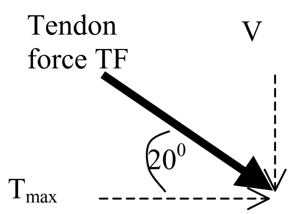
This unbonded length should be verified using the internal stability analyses procedures described in Strom and Ebeling (2002b). The verification process uses limiting equilibrium procedures, which can be performed by simple hand calculations or general-purpose ground slope stability (GPSS) procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to meet internal stability performance requirements (i.e., factor of safety of 1.3 for a safety with economy design).

2.1.8.2 Bonded length of anchors, L_b. The usual practice is for the wall designer to specify the anchor capacity and any right-of-way and easement constraints required of the anchorage system. It is up to the tieback anchor contractor, usually a specialty subcontractor, to propose the type of anchorage system to be used to meet the wall design requirements. A preliminary estimate of the bond length (L_b) required to develop the ground anchors follows.

The horizontal anchor forces T_2 and T_3 are all of equal magnitude and correspond to maximum horizontal anchor force T_{max} (Section 2.1.5). Because the horizontal anchor forces T_1 and T_4 are within 7 percent and 2 percent, respectively, of this T_{max} value, the bond length computations will be made using the tendon force value of T_{max}. The computed bond length will be slightly conservative for anchor tendon 1.

With 6-ft spacing between soldier beams and using T_{max} = 14,824 lb/ft for all anchors, the maximum anchor (tendon) force, TF, is

$$TF = \frac{T_{max} * 6}{\cos 20} = \frac{14824 * 6}{\cos 20} = 94652 \text{ lb} \approx 94.7 \text{ kips}$$



The empirical method used in the following computations for bond length of anchors is for preliminary design purposes. It is up to the tieback anchor contractor, usually a specialty subcontractor, to propose the type of anchorage system to be used to meet the wall design requirements. The final (working load) anchor capacity shall be verified by proof-testing each anchor in the field and performance testing select anchors (Strom and Ebeling 2002b).

It is assumed in the following computations that a straight shaft pressure-treated ground anchor will be used. The interrelationship between the maximum anchor (tendon) force *TF* and the ultimate anchor capacity TF_{ult} is given by

$$TF = \frac{TF_{ult}}{FS}$$

The factor of safety against anchor failure is set equal to 2.0 for tieback walls. Recall that in this design problem, TF is equal to 94.7 kips. By this equation the minimum value of the ultimate tieback anchor capacity TF_{ult} is equal to 189.4 kips.

Rearranging Equation 1.1, the minimum length of anchor bond zone length L_b is given by

$$L_b = \frac{TF_{ult}}{RLT_{ult}}$$

where RLT_{ult} is the ultimate capacity of rate of load transfer (kips per foot).

Using the data contained in Figure 23 in Andersen (1984) the ultimate load-transfer rate RLT_{ult} for loose sand is set equal to 6 kips per lineal ft. The minimum value for L_b is

$$L_b = \frac{189.4}{6} = 31.6 \text{ ft}$$

The computed minimum anchor bond length value is less than 40 ft so the tieback anchorage system is feasible. (Alternatively, a post-grouted (regroutable) ground anchor system may be considered since it is likely to result in a lower L_b value.)

Total anchor lengths ($Lt_1 = L_1 + L_b$).

Top-tier anchor:

$$Lt_1 = 31.8 + 31.6 = 63.4 \text{ ft} \approx 64 \text{ ft}$$

Second-tier anchor:

$$Lt_2 = 26.35 + 31.6 = 57.95 \text{ ft} \approx 58 \text{ ft}$$

Third-tier anchor:

$$Lt_3 = 20.9 + 31.6 = 52.5 \text{ ft} \approx 53 \text{ ft}$$

Lower-tier anchor:

$$Lt_4 = 15.56 + 31.6 = 47.16 \text{ ft} \approx 48 \text{ ft}$$

This total anchor length (bonded + unbonded length) should be verified using the external stability analyses procedures described in Strom and Ebeling (2002b). This verification process also uses limiting equilibrium procedures, which can be performed by simple hand calculations or GPSS procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to prevent ground mass stability failure (i.e., meet external stability performance requirements with factor of safety of 1.3 for a “safety with economy” design).

2.1.9 Anchor strands

The number of 0.6-in.-diam ASTM A416, Grade 270 (American Society for Testing and Materials (ASTM) 1999), strands required to meet "safety with economy" design requirements is determined. It is assumed that the final design force after losses will be based on an allowable anchor stress of 0.6 f_y, or 35.2 kips per strand.

Use the same maximum anchor load TF = 94.7 kips for sizing all four anchor strands (since $T_2 = T_3 = T_{max}$, and T_1 and T_4 are within 7 percent and 2 percent, respectively, of T_{max}).

From Table 8.5, Strom and Ebeling (2001),

Capacity of three 0.6-in. strands = 105.6 > 94.7 kips OK

Use three 0.6-in. strands.

2.1.10 Drilled-in shaft diameter

The drilled-in soldier beam will be fabricated from a pair of MC 10×28.5 shapes (using Grade 50 steel), as discussed in Section 2.1.7. Additionally, a 12-in.-diameter hole, with casing, will be used to construct the anchor bond zone for all anchors, as discussed in Section 2.1.8.2.

The depth, d, and flange width, b_f, of an MC 10×28.5 are

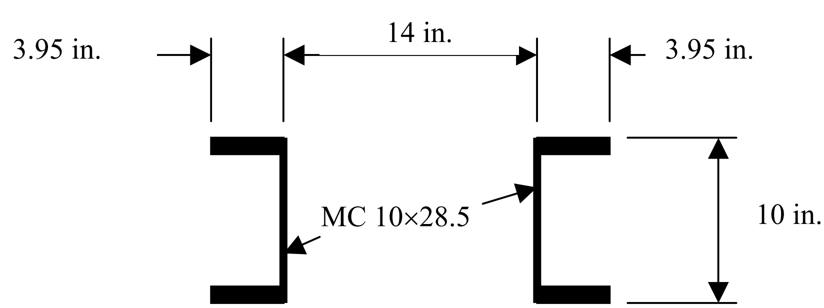
$$d = 10 \text{ in.}$$

and

$$b_f = 3.95 \text{ in.}$$

From Table 8.6, Strom and Ebeling (2001), trumpet diameter for three 0.6-in. strands and Case I corrosion protection = 5-7/8 in.

The distance between channels is set equal to 14 in. to allow ample room for a casing to keep the hole open in the loose sand until the anchor zone grout has been placed. For anchor zone details, see Figure 10.2(b), Strom and Ebeling (2001).



The diameter for the drilled shaft (b) required to install the soldier beams is determined next.

For the previously described configuration of the pair of MC 10×28.5 shapes, the diagonal (from flange tip to flange tip) is given by

$$diagonal = \sqrt{d^2 + (2b_f + clear\ spacing)^2}$$

$$diagonal = \sqrt{(10)^2 + (2 \bullet 3.95 + 14)^2}$$

$$diagonal = \sqrt{(10)^2 + (21.9)^2}$$

$$diagonal = 24.075 \text{ in.}$$

To install the fabricated pair of MC 10×28.5 shapes, the diameter of the drilled shaft (b) must be greater than the flange tip to flange tip diagonal of 24.075 in. Use 26-in.-diameter drilled shaft.

2.1.11 Temporary timber lagging

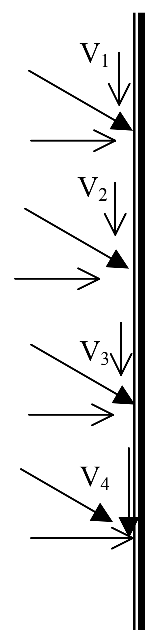
A temporary lagging design based on a uniform soil pressure distribution will be overly conservative since significant soil arching occurs behind soldier beam walls. Therefore, the size of the timber lagging is based primarily on experience or semi-empirical rules (see Table 12 of FHWA-SA-99-015 and Table 8.7 of Strom and Ebeling 2001).

Clear lagging span ≈ soldier beam spacing = 6 ft

From Table 8.7, Strom and Ebeling (2001)

For sands and gravels, recommended thickness = 3 in.

Use 3-in. timber lagging.



2.1.12 Soldier beam toe embedment

Soldier beam toe embedment requirements for both vertical and horizontal loads must be determined. With respect to the vertical component of prestress anchor load:

$$\begin{aligned}\sum V &= \left(V_1 + V_2 + V_3 + V_4\right) \\ &= \left[\left(T_1 + T_2 + T_3\right) * \tan 20° + T_4 * \tan 15°\right] * 6 \\ &= \left[\left(13847 + 14824 + 14824\right) * \tan 20° + 14515 * \tan 15°\right] * 6 \\ &= 118321 \text{ lb} = 118.32 \text{ kips}\end{aligned}$$

FHWA-SA-99-015 (page 95) general design recommendations for concrete backfill of predrilled holes include the use of structural concrete from the bottom of the hole to the excavation base and a lean-mix concrete for the remainder of the hole. The design concept is to provide maximum strength and load transfer in the permanently embedded portion of the soldier beam while providing a weak concrete fill in the upper portion which can be easily removed and shaped to allow lagging installation. However, contractors often propose the use of lean-mix concrete backfill for the full depth of the hole to avoid the delays associated with providing two types of concrete in relatively small quantities. This design example follows the granular soil design examples given in Section 10.1 of FHWA-RD-97-130 and in Example 1 of Appendix A of FHWA-SA-99-015, assuming that lean-mix concrete is used for the full depth of the hole.

The following computations are made to determine total force that the drilled-in shaft foundation must resist. A 14-ft depth of penetration is assumed in these computations for a 50-ft exposed wall height.

The total drilled-in soldier beam shaft weight assuming a 14-ft toe length is equal to the vertical component of anchor force plus the weight of a pair of MC 10×28.5 plus the weight of lean-mix concrete backfill plus the weight of timber lagging. The magnitude of each of these forces is summarized in the following five steps:

a. Vertical component of anchor force = 118.32 kips

b. Weight of 2 MC 10×28.5 channels for 64-ft length = 2*0.0285*64 = 3.65 kips

c. Computation of the weight of lean-mix concrete backfill for a drilled-in soldier beam of length 64 ft:

(1) Weight of lean-mix concrete backfill for a drilled shaft diameter (d_s) of 26 in. and a drilled-in soldier beam cylinder of length 64 ft:

$$\text{Total area} = \pi \bullet \frac{d_s^2}{4}$$

$$\text{Total area} = \pi \bullet \frac{(26)^2}{4}$$

$$\text{Total area} = 530.93\ \text{in.}^2 = 3.687\ \text{ft}^2$$

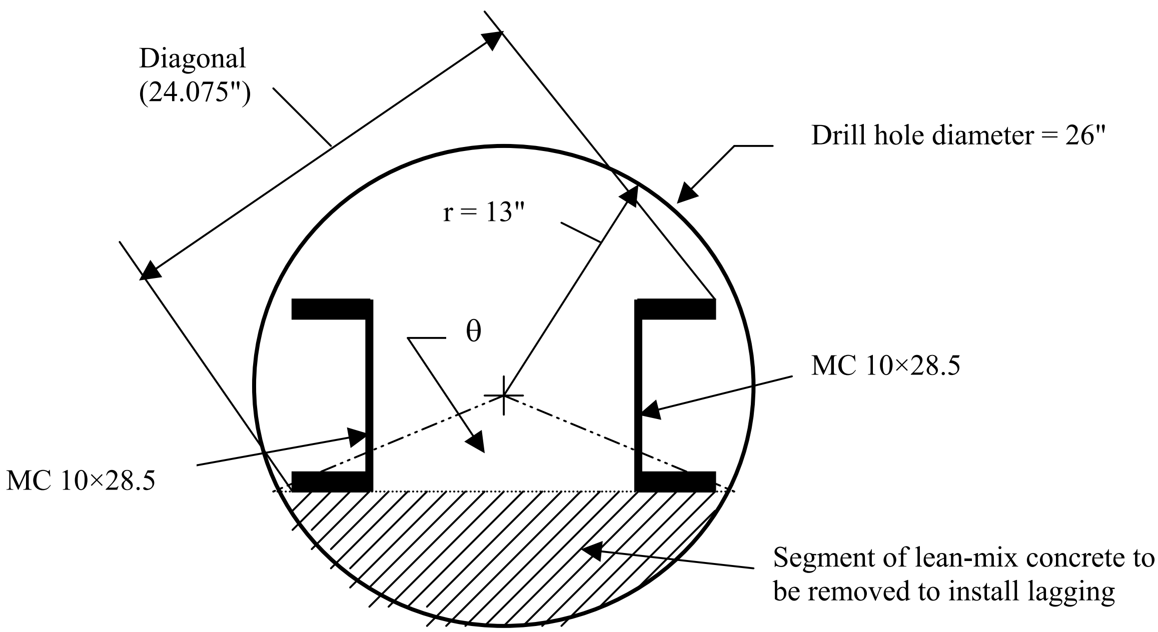
$$\text{Gross weight} = \left(0.145\ \frac{\text{kips}}{\text{ft}^3}\right) \bullet \text{Total area} \bullet 64\ \text{ft}$$

$$\text{Gross weight} = \left(0.145\frac{\text{kips}}{\text{ft}^3}\right) \bullet (3.687) \bullet 64$$

$$\text{Gross weight} = 34.22\ \text{kips}$$

That is, the gross weight of a 64-ft-high, 26-in.-diameter lean-mix concrete cylinder is 34.22 kips. (This does not account for the weight of lean-mix concrete removed when placing the lagging.)

(2) Reduction in gross weight of a 64-ft-long cylinder for removal of the lean-mix concrete backfill during lagging installation. Compute the area of lean-mix concrete to be removed down the 50 ft (exposed) of height in front of the flanges of the pair of MC 10×28.5 shapes:



(3) Computation of the segment (of circle) area in front of flanges to be removed:

$$\theta = 2 \bullet \left[\cos^{-1}\left(\frac{\text{half channel depth}}{\text{radius}}\right)\right] = 2 \bullet \left[\cos^{-1}\left(\frac{5}{13}\right)\right]$$
$$= 134.76 \text{ deg}$$

where

$$\text{half channel depth} = \frac{d}{2} = \frac{10}{2} = 5 \text{ in.}$$

$$r = \text{radius} = \frac{\text{diameter}}{2} = \frac{d_s}{2} = \frac{26}{2} = 13 \text{ in.}$$

$$\text{Segment area} = \pi r^2 \frac{\theta}{360} - r^2 \frac{\sin\theta}{2} = 138.75 \text{ in.}^2 = 0.96 \text{ ft}^2$$

The exposed wall height equals 50 ft. The area of lean-mix concrete to be removed down the 50 ft of exposed wall in front of the flanges for the pair of MC 10×28.5 shapes is equal to 0.963 ft^2 per ft of exposed wall height. For this 26-in.-diameter drilled-in shaft, the area removed represents approximately 26 percent of the cross-sectional area of the original 26-in.-diameter (cylinder) of lean-mix concrete per ft of exposed height.

(4) Computation of the weight of lean-mix concrete removed during placement of lagging over the 50 ft of exposed height of wall:

$$\text{Weight removed} = \left(0.145\,\frac{\text{kips}}{\text{ft}^3}\right) \bullet \text{Segment area} \bullet H$$

$$\text{Weight removed} = \left(0.145\,\frac{\text{kips}}{\text{ft}^3}\right) \bullet 0.963\ \text{ft}^2 \bullet 50\ \text{ft} = 6.98\ \text{kips}$$

(5) Computation of the weight of lean-mix concrete backfill for a drilled-in soldier beam of length 64 ft less the weight removed during placement of lagging:

$$\text{Lean mix net weight} = \text{Gross weight - Weight removed}$$

$$\text{Lean mix net weight} = 34.22 - 6.98 = 27.24\ \text{kips}$$

d. Computation of the weight of timber lagging over 50-ft exposed height for a span of 6 ft:

$$\text{Lagging weight} = \left(0.05\,\frac{\text{kips}}{\text{ft}^3}\right) \bullet \text{span} \bullet \text{Height} \bullet \text{thickness}$$

$$\text{Lagging weight} = \left(0.05\,\frac{\text{kips}}{\text{ft}^3}\right) \bullet 6\ \text{ft} \bullet 50\ \text{ft} \bullet \left(\frac{3}{12}\right) = 3.75\ \text{kips}$$

e. Computation of the applied total drilled-in soldier beam axial load:

$$\text{Q}_{applied} = \sum V + \text{Weight of channels} + \text{Lean mix}$$
$$\text{net weight} + \text{Lagging weight}$$

$$\text{Q}_{applied} = 118.32\ \text{kips} + 3.65\ \text{kips} + 27.24\ \text{kips}$$
$$+ 3.75\ \text{kips} = 152.95\ \text{kips}$$

Thus, for the 26-in.-diameter drilled-in shaft with a 14-ft depth of penetration, the applied axial load is equal to 152.95 kips.

2.1.13 Depth of toe penetration, *D*

This section outlines the depth of penetration computations. For a drilled-in shaft,

Ultimate axial resistance = skin friction resistance + tip resistance

Hence:

$$Q_{ult} = Q_{skin} + Q_{tip} \qquad \text{(see Equation 8.25 in Strom and Ebeling 2001)}$$

The factors of safety for axial capacity of drilled-in soldier beams in granular soil are

$FS_{skin} = 2.0$ and $FS_{skin} = 2.5$

according to Table 8.9 in Ebeling and Strom (2001) and Table 8.9 in FHWA-SA-99-015. Thus, the allowable axial load Q_{all} is given by

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} \qquad \left(\begin{array}{l}\text{modified form of Equation 8.18} \\ \text{in Strom and Ebeling 2001}\end{array}\right)$$

The traditional potential foundation failure mode due to axial loading is based on the assumption of the drilled-in shaft being fully effective in transferring the applied vertical load from the pair of steel channels through the lean-mix concrete mix to the surrounding soil. The corresponding traditional computation assumes the axial capacity is due to the drilled-in shaft acting as a single structural unit within the surrounding granular soil media. FHWA-SA-99-015 (page 95) and FHWA-SA-97-130 (page 90) note that for lean-mix backfilled drilled-in shafts, the lean-mix concrete may not be sufficiently strong to allow vertical load transfer from the soldier beam to the concrete. Consequently, a second potential failure mode must also be considered: The alternative potential failure mode assumes the soldier beam "punches" through the lean-mix, in which case the drilled-in shaft cross-section (assumed to be rectangular) will not be effective in transferring the load to the surrounding soil. Both potential failure modes are evaluated and the smallest capacity is used in the design. These computations are demonstrated in the following two sets of analyses. Note that in each set of computations a different value for depth of penetration D was used. Since the mechanisms for the two potential modes of failures are different, the minimum depth of penetration values required to satisfy the factors of safety against failure will also differ.

2.1.13.1 Analysis 1: Drilled-in shaft capacity (with shaft acting as single unit). In Section 2.1.10 the diameter of the drilled-in shaft was established to be 26 in. By trial and error using the following design analysis procedure, it is determined that a depth of penetration (D) equal to 14 ft is required to meet the aforementioned factor of safety requirements. The soldier beam length is equal to 64 ft (= H + D = 50 ft + 14 ft).

a. *Ultimate skin friction.* The ultimate resistance due to skin friction, Q_{skin}, is given by

$$Q_{skin} = f_{skin} \bullet A_{cylinder}$$

The average unit skin friction is computed using the Strom and Ebeling (2001) Equation 8.26 to be

$$f_{skin} = \beta \bullet \sigma'_{ave} \text{ with the limitation that } f_{skin} < 4 \text{ ksf}$$

$$\beta = 1.5 - 0.135 \bullet \left[\frac{H + D}{2}\right]^{0.5} \text{ with the limitation that } 0.25 < \beta < 1.25$$

$$\beta = 1.5 - 0.135 \bullet \left[\frac{50 + 14}{2}\right]^{0.5} = 0.736$$

FHWA HI-88-042 (1988) stated that β is independent of soil strength because drilling disturbance reduces the friction angle to a common value regardless of initial soil strength.

$$\sigma'_{ave} = \gamma \bullet \left(\frac{H + D}{2}\right) = 115 \bullet \left(\frac{50 + 14}{2}\right) = 3680 \text{ psf}$$

resulting in

$$f_{skin} = 0.736 \bullet 3860 \text{ psf} \bullet \left(\frac{\text{kips}}{1000 \text{ lbf}}\right) = 2.71 \text{ ksf}$$

The surface area of the drilled-in shaft is given by

$$A_{cylinder} = \pi \bullet (diameter) \bullet D$$

$$A_{cylinder} = \pi \bullet \left[26 \text{ in.} \bullet \left(\frac{\text{ft}}{12 \text{ in.}}\right)\right] \bullet 14 \text{ ft} = 95.295 \text{ ft}^2$$

Thus, the ultimate resistance due to skin friction along the 14-ft long depth of penetration of the drilled-in shaft is

$$Q_{skin} = f_{skin} \bullet A_{cylinder} = 2.71 \bullet 95.295 = 258.2 \text{ kips}$$

b. *Ultimate tip resistance.* The ultimate tip resistance due to end bearing, Q_{tip}, is given by

$$Q_{tip} = q_b \bullet A_{tip}$$

The unit end bearing ultimate resistance is computed using the Strom and Ebeling (2001) Table 8.10 relationship:

$$q_b = 1.2 \bullet (\text{uncorrected SPT N - value}) \text{ in units of ksf and with the limitation that the uncorrected SPT N - value be less than 75.}$$

$$q_b = 1.2 \bullet (15) = 18 \text{ ksf}$$

The cross-sectional area of the tip is

$$A_{tip} = \pi \bullet \frac{(diameter)^2}{4}$$

$$A_{tip} = \pi \bullet \frac{\left[26 \text{ in.} \bullet \left(\frac{\text{ft}}{12 \text{ in.}}\right)\right]^2}{4} = 3.687 \text{ ft}^2$$

Thus, the ultimate tip resistance of the 26-in.-diameter drilled-in shaft is

$$Q_{tip} = q_b \bullet A_{tip} = 18 \bullet 3.687 = 66.4 \text{ ksf}$$

c. *Ultimate axial load resistance.* The ultimate axial load Q_{ult} is computed to be

$$Q_{ult} = Q_{skin} + Q_{tip} = 258.2 + 66.4 = 324.6 \text{ kips}$$

Note that skin friction provides 74 percent of the ultimate axial load resistance, while tip resistance due to end bearing provides 24 percent of this ultimate axial load value.

d. *Allowable axial load.* The allowable axial load Q_{all} is computed to be

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} = \frac{258.2}{2.0} + \frac{66.4}{2.5} = 129.1 + 26.56 = 155.66 \text{ kips}$$

Note that skin friction provides 83 percent of the allowable axial load resistance, while tip resistance due to end bearing provides 17 percent of this allowable axial load value.

The allowable axial load Q_{all} for this 26-in.-diameter drilled-in shaft with an assumed 14-ft depth of embedment is 155.66 kips, which is 2.71 kips larger than the applied axial load of 152.97 kips (see Section 2.1.12), i.e., $Q_{applied} > Q_{all}$. Thus, a 14-ft depth of penetration is acceptable for this assumed potential mode of foundation failure. Recall that this potential mode of foundation failure assumes the shaft acting as single unit.

Solution process: The procedure used to determine the depth of penetration D in this problem was as follows: assume a depth of penetration; compute the ultimate axial load resistance Q_{ult}; compute the allowable axial load Q_{all}; compute the total applied axial load for the drilled-in soldier beam system $Q_{applied}$ (Section 2.1.12); adjust the depth of penetration D as necessary; and repeat computations until Q_{all} is approximately equal to $Q_{applied}$. Ensure that for the final value of D used in the computations, Q_{all} is greater than or equal to $Q_{applied}$.

2.1.13.2 Analysis 2: "Punching" soldier beams capacity. The drilled-in soldier beam backfilled with a lean-mix concrete has the potential not to act as a single structural unit in which the axial load is transferred from the pair of channels through the lean-mix to the surrounding soil foundation but where the pair of channels "punches" through the lean-mix concrete backfill. In Section 2.1.10 the diameter of the drilled-in shaft was established to be 26 in. By trial and error using the following design analysis procedure, it is determined that a depth of penetration (D) equal to 10 ft is required to meet established factor of safety requirements. (Note that this value of D differs from the 14-ft value used in Analysis 1 computations). The authors of this report are demonstrating that the minimum required depth of penetration is not the same for the two different types of failure modes). For the Analysis 2 procedure the soldier beam length is equal to 60 ft (= H + D = 50 ft + 10 ft). As an alternative the designer could verify the depth of penetration established by the design Analysis 1 procedure satisfies factor of safety requirements for the Analysis 2 procedure.

The following computations assume the pair of soldier beam channels will punch through the lean-mix concrete backfill rather than transfer the load through the backfill to the ground. Note that in these computations the diameter of the drilled shaft is not used (i.e., the drilled shaft cross-section will not be effective in transferring the load to the surrounding soil). Instead, the rectangular "box" perimeter of the pair of channels is used in both the skin friction and tip resistance computations.

a. Ultimate skin friction. The ultimate resistance due to skin friction, Q_{skin}, is given by

$$Q_{skin} = f_{skin} \bullet A_{box}$$

The average unit skin friction for this "punching" mode of failure is computed using the Strom and Ebeling (2001) Equation 8.20 to be

$$f_{skin} = K \bullet \sigma'_{ave} \bullet \tan(\delta)$$

with

$$\sigma'_{ave} = \gamma \bullet \left(\frac{H + D}{2}\right) = 115 \bullet \left(\frac{50 + 10}{2}\right) = 3450 \text{ psf}$$

FHWA-RD-97-130 (page 94) and FHWA-SA-99-015 (page 95) note that when a lean-mix concrete backfill is used in a drilled-in shaft, f_{skin} is computed using K = 2 and δ = 35 degrees in the f_{skin} equation (see page 180 in Strom and Ebeling 2001). Note that these parameter values are specific to the "punching" mode of failure through the lean-mix concrete. Thus, f_{skin} becomes

$$f_{skin} = 2 \bullet \left[3450 \text{ psf} \bullet \left(\frac{\text{kips}}{1000 \text{ lb}} \right) \right] \bullet \tan(35) = 4.831 \text{ ksf}$$

The surface area of the rectangular "box" defined by the perimeter of the pair of channels is given by

$$A_{box} = [2 \bullet (\text{channel depth}) + 2 \bullet (\text{flange - to - flange width})] \bullet D$$

$$A_{box} = \left[2 \bullet (\text{channel depth}) + 2 \bullet \begin{pmatrix} 2 \bullet b_f + \text{clear space} \\ \text{between channels} \end{pmatrix} \right] \bullet D$$

$$A_{box} = [2 \bullet (10 \text{ in.}) + 2 \bullet (2 \bullet 3.95 \text{ in.} + 14 \text{ in.})] \bullet \left(\frac{1}{12} \right) \bullet 10 \text{ ft}$$

$$= [(20 \text{ in.}) + (43.8 \text{ in.})] \bullet \left(\frac{1}{12} \right) \bullet 10 \text{ ft} = 53.167 \text{ ft}^2$$

Thus, the ultimate resistance due to skin friction along the 10-ft-long depth of penetration of the drilled-in shaft is

$$Q_{skin} = f_{skin} \bullet A_{box} = 4.831 \text{ ksf} \bullet 53.167 = 256.87 \text{ kips}$$

b. *Ultimate tip resistance*. The ultimate tip resistance due to end bearing, Q_{tip}, is given by

$$Q_{tip} = q_b \bullet A_{tip}$$

The unit end bearing ultimate resistance is computed using the Strom and Ebeling (2001) Equation 8.21 relationship

$$q_b = \sigma'_v \bullet N_q$$

where

$$\sigma'_v = \gamma \bullet D$$

$$\sigma'_v = 115 \bullet 10 = 1150 \text{ psf}$$

According to FHWA-RD-97-130 (page 94), a value of N_q in the middle range recommended by Meyerhoff give the best estimate of the end bearing capacities. Using Figure 8.11 in Strom and Ebeling (2001), this mid-range Meyerhoff N_q value is equal to 40 for ϕ equal to 30 degrees. This results in a unit end bearing ultimate resistance equal to

$$q_b = 1150 \bullet \left(\frac{1}{1000} \right) \bullet 40 = 46 \text{ ksf}$$

The cross-sectional area of the rectangular "box" tip is

$$A_{tip} = (\text{channel depth}) \bullet (\text{flange - to - flange width})$$

$$A_{tip} = (\text{channel depth}) \bullet \left(2 \bullet b_f + \text{clear space between channels}\right)$$

$$A_{tip} = [(10 \text{ in.}) \bullet (2 \bullet 3.95 \text{ in.} + 14 \text{ in.})] = [(10 \text{ in.}) \bullet (21.9 \text{ in.})] = 219 \text{ in.}^2$$

Thus, the ultimate tip resistance of the 26-in.-diameter drilled-in shaft is

$$Q_{tip} = q_b \bullet A_{tip} = 46 \bullet 219 \bullet \left(\frac{1}{144} \right) = 69.96 \text{ ksf}$$

c. *Ultimate axial load resistance.* The ultimate axial load Q_{ult} is computed to be

$$Q_{ult} = Q_{skin} + Q_{tip} = 256.87 + 69.96 = 326.83 \text{ kips}$$

Note that skin friction provides 79 percent of the ultimate axial load resistance, while tip resistance due to end bearing provides 21 percent of this ultimate axial load value.

d. *Allowable axial load.* The allowable axial load Q_{all} is computed to be

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} = \frac{256.87}{2.0} + \frac{69.96}{2.5} = 128.44 + 27.98$$

$$= 156.42 \text{ kips}$$

Note that skin friction provides 82 percent of the allowable axial load resistance, while tip resistance due to end bearing provides 18 percent of this allowable axial load value.

The allowable axial load Q_{all} for this 26-in.-diameter drilled-in shaft with an assumed 10-ft depth of embedment is 156.42 kips, which is 5.84 kips larger than the applied axial load of 150.58 kips (computations not shown but follow those made in Section 2.1.12 using a 10-ft depth of penetration). Thus, a 10-ft depth of penetration is acceptable for this assumed potential mode of foundation failure. Recall that this potential mode of foundation failure assumes the soldier beam "punches" through the lean mix concrete backfill.

Solution process: The procedure used to determine the depth of penetration D in this problem was as follows; assume a depth of penetration; compute the ultimate axial load resistance Q_{ult}; compute the allowable axial load Q_{all}; compute the total applied axial load for the drilled in solder beam system $Q_{applied}$ (following the procedure outlined in Section 2.1.12); adjust the depth of penetration D as necessary and repeat computations until Q_{all} is approximately equal to $Q_{applied}$. Ensure that for the final value of D used in the computations, that Q_{all} is greater than or equal to $Q_{applied}$.

2.1.13.3 Concluding remarks. The minimum required depths of penetration were computed in this subsection for two potential failure modes. It was found in design Analysis 1 that a 14-ft minimum depth of penetration is required to be safe by the traditional potential foundation failure mode in which the drilled-in shaft acts as a single structural unit within the surrounding granular soil media. It was found in design Analysis 2 that a 10-ft minimum depth of penetration is required for the system to be safe against the alternative potential failure mode that assumes the soldier beam "punches" through the lean-mix. Therefore, the required depth of penetration D for this drilled-in shaft retaining structure is 14 ft.

2.1.14 Lateral capacity of soldier beam toe

Assume, based on vertical load requirements, that the final toe penetration (D) is 14 ft.

Check lateral capacity of soldier beam toe:

Subgrade reaction per foot of wall, R = 2,780 lb/ft (Section 2.1.6)

Total toe reaction = 2,780*6= 16,680 lb = 16.7 kips

A spreadsheet incorporating the Wang-Reese passive resistance equations (Table 2.1) is used to determine lateral resistance of the soldier beam toe following the procedure outlined in Section 8.7 of Strom and Ebeling (2001) or Section 6.2 in FHWA-RD-97-130. Note that the soldier beam width of 1.825 ft (21.9 in.) is used for drilled shafts backfilled with lean concrete as per FHWA-RD-97-130 (page 111) recommendations. If structural concrete is used to backfill the shaft, then the drilled shaft diameter (= 26 in.) would be used in the computations. Alpha and beta in this table are angles used to define the three-dimensional geometrical configuration of the "passive" failure wedge developing in front of the soldier beam on the excavated side (refer to Figure 8.6 in Strom and Ebeling 2001). The Wang-Reese definitions are

$$\beta = 45 + \frac{\phi'}{2} = 45 + \frac{30}{2} = 60\,\text{deg}$$

Table 2.1
Spreadsheet for Computing Passive Resistance for Sand ("Safety with Economy" Design)

INPUT VARIABLES

unit weight(kip/ft^3)	wall height(ft)	beam width(ft)	beam spacing(ft)	phi	alpha	beta	toe depth(ft)	toe reaction(kip)
0.115	**50**	**1.825**	**6**	**30**	**10**	**60**	**14**	**16.7**

COMPUTED FUNCTIONS

tangent phi	tan beta	tan(beta-phi)	tan alpha	sin beta	cos alpha	sine phi	cos phi	cos beta	Ko	Ka	Kp	Sc
0.577	1.7321	0.5774	0.1763	0.8660	0.9848	0.5000	0.8660	0.5000	0.5000	0.3329	2.9927	6

SPREADSHEET FOR EVALUATING TOE CAPACITY FOR SAND ("safety with economy" design)

						(Eq 6.15)			(Eq 6.17)					
			(Eq 6.13)	*wedge*	*wedge*	*intersecting*	(Eq 6.16)		*Rankine*		*total*	*total*	*net*	
Toe		(Eq 6.14)	*wedge*	*resistence*	*resistance*	*wedge*	*flow*	*critical*	*passive*	*passive*	*passive*	*active*	*passive*	
depth	*wing*	*interaction*	*resistance*	*D=di*	*D=di, a=0*	*resistance*	*resistance*	*resistance*	*resistance*	*resistance*	*force*	*force*	*force*	*factor*
(D)	*width*	*height, di*	*(kip/ft)*	*(kip/ft)*	*(kip/ft)*	*(kip/ft)*	*(kip/ft)*	*(kip/ft)*	*(kip/ft)*	*(kip/ft)*	*(kip)*	*(kip)*	*(kip)*	*of safety*
col.1	*col 2*	*col 3*	*col 4*	*col 5*	*col 6*	*col 7*	*col 8*	*col 9*	*col 10*	*col 11*	*col 12*	*col 13*	*col 14*	*col 15*
0	0.0000	-9.8229	0.0000	11.9713	7.3888	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.9143	-1.9143	-0.1146
1	0.3054	-8.8229	0.8178	9.0924	5.3955	0.8178	5.9587	0.8178	2.6930	0.8178	0.4089	1.9335	-1.5246	-0.0913
2	0.6108	-7.8229	2.0119	6.5898	3.6835	2.0119	11.9174	2.0119	5.3860	2.0119	1.8238	3.9052	-2.0815	-0.1246
3	0.9162	-6.8229	3.5824	4.4636	2.2528	3.5824	17.8762	3.5824	8.0790	3.5824	4.6209	5.9152	-1.2943	-0.0775
4	1.2216	-5.8229	5.5292	2.7137	1.1035	5.5292	23.8349	5.5292	10.7721	5.5292	9.1767	7.9636	1.2131	0.0726
5	1.5270	-4.8229	7.8523	1.3402	0.2355	7.8523	29.7936	7.8523	13.4651	7.8523	15.8674	10.0502	5.8173	0.3483
6	1.8325	-3.8229	10.5518	0.3430	-0.3511	10.5518	35.7523	10.5518	16.1581	10.5518	25.0695	12.1751	12.8944	0.7721
7	2.1379	-2.8229	13.6276	-0.2779	-0.6564	13.6276	41.7111	13.6276	18.8511	13.6276	37.1592	14.3382	22.8209	1.3665
8	2.4433	-1.8229	17.0797	-0.5225	-0.6803	17.0797	47.6698	17.0797	21.5441	17.0797	52.5128	16.5397	35.9731	2.1541
9	2.7487	-0.8229	20.9082	-0.3907	-0.4228	20.9082	53.6285	20.9082	24.2371	20.9082	71.5067	18.7795	52.7273	3.1573
10	3.0541	0.1771	25.1130	0.1174	0.1159	25.1115	59.5872	25.1115	26.9302	25.1115	94.5166	21.0575	73.4591	4.3987
11	3.3595	1.1771	29.6941	1.0019	0.9361	29.6283	65.5459	29.6283	29.6232	29.6232	121.8839	23.3738	98.5101	5.8988
12	3.6649	2.1771	34.6516	2.2627	2.0376	34.4265	71.5047	34.4265	32.3162	32.3162	152.8536	25.7284	127.1251	7.6123
13	3.9703	3.1771	39.9854	3.8998	3.4204	39.5060	77.4634	39.5060	35.0092	35.0092	186.5163	28.1213	158.3949	9.4847
14	**4.2757**	**4.1771**	**45.6955**	**5.9133**	**5.0846**	**44.8669**	**83.4221**	**44.8669**	**37.7022**	**37.7022**	**222.8720**	**30.5525**	**192.3195**	**11.5161**

and

$$\alpha = \frac{\phi'}{3} \text{ to } \frac{\phi'}{2} \text{ for loose sands, and } \phi \text{ for dense sands}$$

$$\alpha = \frac{30}{3} \text{ to } \frac{30}{2} = 10 \text{ to } 15 \text{ deg, use } 10 \text{ deg}$$

Passive resistance back-calculated for soldier beam and lagging systems compares favorably with passive resistance equations developed by Wang and Reese (1986). Several passive failure mechanisms must be evaluated for each increment of soldier beam embedment, and the pressure associated with the governing failure condition summed over each increment of depth to determine the soldier beam total passive resistance. This process is summarized for the cohesionless soil example in Table 2.1 for the safety with economy design (and in Section 2.2.13 for the stringent displacement control design). In Table 2.1, the pressures attributed to the various failure mechanisms are provided in columns 5 through 8, and the pressures associated with the governing failure condition are indicated in column 9. The equation numbers referenced in the various columns of the table refer to equations from FHWA-RD-97-130. Similar equations can be found in FHWA-SA-99-015 and in Strom and Ebeling (2001). Table 2.2 provides the reference equation number associated with each of these three references.

The computations summarized in Table 2.1 are for the 50-ft-high tieback wall in granular soil (sand) for the safety with economy design. These computations explicitly follow those given in the FHWA spreadsheet procedure (FHWA-RD-97-130, Figure 98, page 192 for a two-tier, drilled-in soldier beam wall). It should be noted that the FHWA "Granular Soil Design Example" is for a 30-ft-high tieback wall (FHWA-RD-97-130, Section 10.1, page 171). The differences in toe passive resistance (i.e., Table 2.1 herein versus FHWA-RD-97-130, Figure 98) are due to the soldier beam width of 1.825 ft in Table 2.1 versus 1.067 ft in FHWA report) and the soil properties (ϕ = 30 deg, γ = 115 psf in Table 2.1 versus ϕ = 29 deg, γ = 108 psf in FHWA report). The total active force and net passive resistance (columns 13 and 14, respectively) are dependent on wall height (30 ft for the FHWA example versus 50 ft for the Table 2.1 example). In accordance with FHWA-RD-97-130, Table 2.1 includes a total active force reduction (column 13) to account for the active soil pressures acting on the toe of the soldier beam. The total net passive force (i.e., toe passive resistance minus toe active soil pressure) is indicated in column 14 of Table 2.1.

The factor of safety is indicated in column 15. Table 2.1 indicates that soldier beams spaced at 6 ft on centers with a 14-ft toe penetration will have a lateral resistance of 192.32 kips. This provides a factor of safety of 11.5, which is greater than the minimum of 1.5 required for a safety with economy design.

A summary of the results for 50-ft-high soldier beam safety with economy design is given in Table 2.3.

Table 2.2 Equation References for Passive Resistance Calculations Stated in Tables 2.1 and 2.4				
		Reference Document–Equation Number		
Column No.	Description of Equation	FHWA-RD-97-130	FHWA-SA-99-015	Strom and Ebeling (2001)
3	Intersection depth of intersecting failure wedges	Eq. 6.14	Eq. B-3	Eq. 8.8
7	Intersecting wedge resistance	Eq. 6.15	Eq. B-4	Eq. 8.9
8	Flow resistance	Eq. 6.16	Eq. B-5	Eq. 8.10
10	Rankine passive resistance	Eq. 6.17	Eq. B-6	Eq. 8.11

2.1.15 Failure planes below the bottom of the cut

Since most cohesionless soils exhibit friction angles greater than 30 degrees, the difference between the total load required to stabilize the cut for failure planes that pass through the corner of the cut versus failure planes that pass beneath the bottom of the cut is typically minor according to FHWA-RD-98-065. However as friction angles drop below 30 degrees, the difference becomes significant, with the total load obtained from the evaluation of failure planes that pass beneath the bottom of the cut being greater than from those that pass through the corner of the cut (FHWA-RD-98-065). For loose sands the failure surface may extend below the bottom of the cut thereby increasing the total load required to stabilize the cut. A GPSSP analysis can evaluate failure planes passing below the bottom of the cut. The Spencer method considers both force and moment equilibrium. Therefore it is often selected for the GPSSP analysis of cohesionless soil sites. The Spencer method can be used to determine the total load the tieback system must carry to meet internal stability factor of safety requirements established for the project. The total load determined from a Spencer method internal stability limiting equilibrium analysis can be redistributed into an apparent pressure diagram. This apparent pressure diagram should be used as a basis for the design if it provides a greater total load than that obtained from a conventional apparent pressure diagram (one that assumes a "bottom corner of the cut" failure plane condition). GPSSP analyses are described in FHWA-RD-98-065 and in Strom and Ebeling (2002b). GPSSP analyses should always be used to verify that the total load required to meet internal stability safety requirements is equal to or less than that used for the original design.

As the soil above the failure plane attempts to move out, shear resistance is mobilized in the soldier beams. Shear in the soldier beams provides additional resistance to soil movement. Predicting soldier beam shear requires the consideration of three possible failure modes: (1) shear in the soldier beam; (2) flow of the soil between the soldier beams; and (3) lateral capacity of the soldier beams. These three failure mechanisms are discussed in Section 4.3.3 of FHWA-RD-98-065. The resistance provided by the soldier beams for each of the

Table 2.3
Summary of Results for Four-Tier, 50-ft Drilled-In Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Retaining Granular Soils–"Safety with Economy" Design

Parameter		Value
Wall height		50 ft
Soldier beam spacing		6 ft
Soldier beam design moment		97.6 kip-ft
Soldier beam size		2 MC10 × 28.5
Soldier beam length		64 ft
Drill shaft diameter		26 in.
Toe reaction		16.7 kips
Top-tier anchor	H_1	7 ft, 0 in.
	Anchor inclination	20 deg
	Design load	88.4 kips
	Unbonded length	31.8 ft
	Bonded length	31.6 ft
	Total length	64 ft
	Tendon size	three 0.6-in. strands
Second-tier anchor	H2	10 ft, 9 in.
	Anchor inclination	20 deg
	Design load	94.7 kips
	Unbonded length	26.35 ft
	Bonded length	31.6 ft
	Total length	58 ft
	Tendon size	three 0.6-in. strands
Third-tier anchor	H3	10 ft, 9 in.
	Anchor inclination	20 deg
	Design load	94.7 kips
	Unbonded length	20.9 ft
	Bonded length	31.6 ft
	Total length	53 ft
	Tendon size	three 0.6-in. strands
Lower-tier anchor	H4	10 ft, 9 in.
	Anchor inclination	15 deg
	Design load	90.2 kips
	Unbonded length	15.56 ft
	Bonded length	31.6 ft
	Total length	48 ft
	Tendon size	three 0.6-in. strands

three possible failure modes can be estimated and included in a GPSSP analysis, provided the GPSSP used is capable of modeling the soldier beams as reinforcement.

The calculated mass stability (i.e., external stability) slip circles for loose sands can be deep and located beyond or at the end of the usual tieback anchorage location. Ground mass stability in loose sands can be improved by extending the length of the tiebacks. The use of GPSSP analyses for determining the required position of the back of the tieback anchor is covered in Chapter 4 of FHWA-RD-98-065 and in Strom and Ebeling (2002b).

2.2 "Stringent Displacement Control" Design

For a Corps of Engineers' "stringent displacement control" design, a limiting equilibrium approach is used with a factor of safety of 1.5 applied to the shear strength of the soil. The total earth pressure load (P_{tl}) is then determined based on the limiting equilibrium analysis. Limiting equilibrium calculations for the "stringent displacement control" design are provided below. This process produces an EPF equal to 27.0 pcf, compared with an EPF of 24.3 pcf determined by the previous limiting equilibrium analysis for the safety with economy design (Section 2.1). The total earth pressure load is determined assuming the shear strength of the soil is factored by the target factor of safety such that

$$\phi_{mob} = \tan^{-1}(\tan\phi / FS)$$

Accordingly, for the stringent displacement control design,

$$\phi_{mob} = \tan^{-1}\left(\frac{\tan\phi}{1.5}\right)$$

$$= \tan^{-1}\left(\frac{\tan 30°}{1.5}\right) = 21.05°$$

$$K_A = \tan^2\left(45° - \frac{\phi_{mob}}{2}\right) = 0.471$$

$$P_{tl} = K_A \gamma \frac{H^2}{2} = 0.471 * 115 * \frac{50^2}{2} = 67706 \text{ lb/ft}$$

$$\text{Effective pressure factor, EPF} = \frac{P_{tl}}{H^2} = \frac{67706}{50^2} = 27 \text{ lb/ft}^3$$

2.2.1 Anchor points

One of the intended purposes of installing a tieback wall is to restrict wall and retained soil movements during excavation to a tolerable movement so that adjacent structures will not experience any distress. If a settlement-sensitive structure is founded on the same soil used for supporting the anchors, a tolerable ground surface <u>settlement</u> may be less than 1/2 in. according to FHWA-RD-81-150. FHWA-RD-81-150 also states that if the adjacent structure has a deep foundation that derives its capacity from a deep bearing stratum not influenced by the excavation activity, settlements of 1 in. or more may be acceptable. Obviously, this guidance is geared towards situations involving buildings that are adjacent to the excavation. Figure 75 in FHWA-RD-97-130 gives settlement profiles/envelopes behind flexible walls in different soils.

Wall and retained soil movements predictions are based on experience. Several types of movements are associated with flexible anchored walls. These are described on page 120 of FHWA-SA-99-015. Movement can occur due to (1) wall cantilever action associated with installation of the first anchor; (2) wall bulging actions associated with subsequent excavation stages and anchor installations; (3) wall settlement associated with mobilization of end bearing; (4) elastic elongation of the anchor tendons associated with a load increase; (5) anchor yielding or load redistribution in the anchor bond zone; and (6) mass ground movements behind the tieback anchors. The last three components of deformation result in translation of the wall and are generally small for anchored walls constructed in competent soils according to FHWA-SA-99-015. Typical lateral and horizontal movements for flexible retaining walls have been presented by Peck (1969), FHWA-RD-75-128 (1976), and Clough and O'Rourke (1990). FHWA-RD-97-130 states that maximum lateral movements in ground suitable for permanent ground anchor walls are generally less than 0.005H, with average maximum movements of about 0.002H. For a 50-ft-high wall the <u>average maximum horizontal</u> movement would be 1.2 in. by this relationship. FHWA-RD-97-130 also states that maximum vertical settlements in ground suitable for permanent ground anchor walls are generally less than 0.005H, with average maximum settlement tending toward 0.0015H. Maximum settlement occurs near the wall. For a 50-ft-high wall the <u>average maximum settlement</u> would be 0.9 in. by this relationship. Note that actual wall performance and especially horizontal and vertical deformations are a function of both design and construction details.

Lateral wall movements and ground settlements cannot be eliminated for flexible tieback walls. However, they can be reduced by (1) controlling soldier beam bending deformations (i.e., cantilever and bulging displacements); (2) minimizing soldier beam settlements by installing the tieback anchors at flat angles (note that grouting of anchors installed at angles less than 10 degrees from horizontal is not common unless special grouting techniques are used) and properly designing the embedded portion of the wall to carry applied axial loads; and (3) increasing the magnitude of the anchor design forces for which the anchors are prestressed to over that obtained in a "safety with economy" design (given in Section 2.1).

Among the factors contributing to bending deformations are (1) the depth of excavation prior to installation and prestress of the first row of anchors, and

(2) the span between the subsequent, lower rows of anchors. FHWA-RD-97-130 and others observe that reducing the distance to the upper ground anchor will reduce the cantilever bending deformations. The magnitude of this deformation, which occurs prior to installation of the first row of anchors, increases as the depth of excavation to the upper ground anchor increases. This deformation is often a significant contributor to total wall permanent deformations. Additional displacement constraints are invoked by reducing the span between the ground anchors, which will reduce the bulging deformations. The relationships developed by FHWA-RD-98-067 are recommended in a "displacement control" design procedure given in FHWA-RD-97-130 (page 147) and have been adopted for use in this report. Specifically, the FHWA-RD-97-130 Equation 9.1 is used to estimate cantilever displacement y_c and Equation 9.2 is used to used to estimate bulging deformations y_b and will be given subsequently. The designer sets project-specific horizontal displacement limitations, which, in turn, are set as limiting values for y_c and y_b. The first row anchor depth and spacings for the subsequent rows of anchors are then established that meet this project-specific displacement performance objective. A subsequent example calculation will demonstrate this procedure. On page 148 of FHWA-RD-97-130 the designer is cautioned that movements estimated from these two equations show trends, and they can be used to evaluate the impact of different ground anchor locations. They represent <u>minimum</u> movements that might be expected.

The third distinguishing aspect of the "stringent displacement control" design procedure is the factor of safety used in the EPF computation, set equal to 1.5 as compared with the 1.3 value used in the "safety with economy" design procedure. For this 50-ft-high wall problem, the EPF now becomes 27.0 pcf, which is 11 percent greater than the 24.3-pcf EPF value used in Section 2.1 "safety with economy" tieback wall design. Recall the EPF value will scale the apparent earth pressure diagram used to compute the horizontal design anchor forces, designated as variable T_i in this report (where the subscript i designates the anchorage row number). It is inferred that by using a factor of safety equal to 1.5 in the development of apparent pressure diagram, nearer to at-rest conditions (versus active earth pressure conditions) will occur behind the wall, which along with smaller distance to upper ground anchor and closer anchor spacings, will contribute to reduce wall displacements over a "safety with economy" design. When displacement control of flexible tieback walls is a key consideration the reader is referred to helpful discussions contained within Section 9.1 of FHWA-RD-97-130; Section 2.1.3 of FHWA-RD-81-150; Section 5.11.1 in FHWA-SA-99-015. It should be recognized, however, that where displacement is important to project performance, NLFEM-SSI analysis might be required to properly assess displacement performance. Additional information on NLFEM analysis can be found in Strom and Ebeling (2001). Alternatively, stiff tieback walls should always be considered in those situations where the magnitude of flexible tieback wall deformations (cantilever, bulging, and/or cumulative/final displacements) is of concern (see Strom and Ebeling 2001 or Strom and Ebeling 2002a).

Displacement limits are project specific. For this particular 50-ft-high wall design example, a maximum lateral wall displacement of 0.7 in. will be established for the Mueller et al. (1996) cantilever displacement y_c and the bulging deformation y_b equations.

With the minimum number of four rows of anchors, the vertical anchor spacing from the safety with economy design is as follows:

H_1 = 7 ft, 0 in.

and

$H_2 = H_3 = H_4 = H_5$ = 10 ft, 9 in.

These anchor spacings will be evaluated using Equations 9.1 and 9.2 of FHWA-RD-97-130 to determine if the associated cantilever and interior spans can be used to meet stringent displacement control performance requirements.

Approximate cantilever deformation, y_c , allowing 1.5 ft overexcavation for placement of top anchor, h_1 = 7 + 1.5 = 8.5 ft, with E_s =3,000 psi for loose sand, and K_o = 0.5,

$$y_c = \frac{4 \bullet K_o \bullet \gamma \bullet (h_1)^2}{E_s} = \frac{4 \bullet 0.5 \bullet 115 \bullet (8.5)^2}{3000 \bullet 12}$$

$$= 0.46 \text{ in.} < 0.7 \text{ in.} \quad \text{OK}$$

The soil modulus (E_s) was obtained from Table 20 of FHWA-RD-97-130.

Approximate span bulging deformation, y_s , with span h = 10.75 ft and wall height H = 50 ft,

$$y_b = \frac{0.8 \bullet K_o \bullet \gamma \bullet h \bullet H}{E_s} = \frac{0.8 \bullet 0.5 \bullet 115 \bullet 10.75 \bullet 50}{3000 \bullet 12}$$

$$= 0.69 \text{ in.} < 0.7 \text{ in.} \quad \text{OK}$$

Anchor spacing satisfies the cantilever and bulging deformation constraints of not greater than 0.7 in. by the Mueller et al. (1998) equations. Note that no constraints on total (i.e., post-construction) horizontal and vertical wall deformations were considered in these computations. Recall that FHWA-RD-97-130 relationships for <u>average</u> maximum horizontal displacements and <u>average</u> maximum settlement (assuming good construction practice in conjunction with good design) may be on the order of 1.2 in. and 0.9 in., respectively. For displacement-sensitive projects, NLFEM analysis of the flexible wall is recommended. Alternatively, a stiff tieback wall system may be considered.

2.2.2 Apparent earth pressure

Referring to the calculations presented previously (in Section 2.1.3), the effective earth pressure (p_e) for the stringent displacement control design becomes

$$p_e = \frac{\text{Total earth pressure load } (P_{tl})}{H - \frac{H_1}{3} - \frac{H_5}{3}}$$

$$= \frac{67706}{50 - \frac{7}{3} - \frac{10.75}{3}} = 1536 \text{ psf}$$

2.2.3 Bending moments on soldier beams

Referring to the Section 2.1.4 calculations, the cantilever bending moment (M_1) and interior span moments (MM_1) are determined for the stringent displacement control design as follows:

$$M_1 = \frac{13}{54} H_1^2 p = \frac{13}{54} * 7^2 * 1536 = 18119 \text{ lb} - \text{ft/ft}$$

and,

$$MM_{(1,2,3)} = \frac{1}{10} (\text{larger of } H_{(2,3,4)})^2 \; p$$

$$= \frac{1}{10} (10.75)^2 (1536)$$

$$= 17750 \text{ lb} - \text{ft/ft}$$

hence,

Maximum moment M_{max} = 18119 lb – ft/ft

2.2.4 Subgrade reaction using tributary area method

Referring to the Section 2.1.6 calculations, the subgrade reaction (R) is determined for the stringent displacement control design as follows:

$$R = \left(\frac{3}{16} H_5 \right) p$$

i.e.,

$$R = \frac{3}{16} * 10.75 * 1536$$

$$= 3096 \text{ lb/ft}$$

2.2.5 Ground anchor load horizontal components

Referring to the Section 2.1.6 calculations, the horizontal component of each tier of anchors is determined for the stringent displacement control design as follows. Assume soldier beam spacing (s) of 6 ft.

Top tier:

$$
\begin{aligned}
T_1 &= \left(\frac{2}{3}H_1 + \frac{1}{2}H_2\right)p \\
&= \left(\frac{2}{3}*7 + \frac{1}{2}*10.75\right)1536 \\
&= 15424 \text{ lb/ft}
\end{aligned}
$$

(Design anchor force = 15.424 kips/ft × 6 ft/cos 20° = 93.5 kips)

Tiers 2, 3:

$$
\begin{aligned}
T_2 = T_3 &= \frac{1}{2}\left(H_2 + H_3\right)p \\
&= \frac{1}{2}\left(10.75 + 10.75\right)1536 \\
&= 16512 \text{ lb/ft}
\end{aligned}
$$

(Design anchor force = 16.512 kips/ft × 6 ft/cos 20° = 105.4 kips)

Lower tier:

$$
\begin{aligned}
T_4 &= \left(\frac{H_4}{2} + \frac{23}{48}H_5\right)p \\
&= \left(\frac{10.75}{2} + \frac{23*10.75}{48}\right)1536 \\
&= 16168 \text{ lb/ft}
\end{aligned}
$$

(Design anchor force = 16.168 kips/ft × 6 ft/ cos 15° = 100.4 kips)

Use T_{max} = 16512 lb/ft

2.2.6 Soldier beam size

Assume soldier beam spacing (s) = 6.0 ft.

Note the minimum permissible spacing is 4.0 ft (see Figure 8.5, Strom and Ebeling 2001).

Hence, the maximum soldier beam design moment ($\underline{M_{max}}$) for the stringent displacement control design is

$$\text{Maximum design moment } \left(M_{max}\right) = \frac{18119}{1000} * 6 = 108.7 \text{ ft} - \text{kip}$$

In accordance with Corps criteria (HQUSACE 1991), the allowable stresses for the soldier beams and wales shall be as follows:

Bending (i.e., combined bending and axial load): $f_b = 0.5\, f_y$

Shear $f_v = 0.33\, f_y$

Allowable stresses are based on 5/6 of the AISC-ASD recommended values (AISC 1989) and reflect the Corps' design requirements for steel structures. Thus,

The allowable bending stress for Grade 50 steel: $F_b = 0.5\, F_y = 25$ ksi

Allowable shear stress for Grade 50 steel: $F_v = 0.33\, F_y = 16.5$ ksi

The required section modulus (S) for the stringent displacement control design using Grade 50 steel is

$$S = \frac{M_{max}}{F_b} = \frac{108.7 * 12}{25} = 52.2 \text{ in.}^3$$

From AISC (1989), HP 10×57 provides $S_{xx} = 58.8 > 52.2$ in.^{3} OK

or,

2 MC 10×33.6 provides $S_{xx} = 55.6 > 52.2$ in.^{3} OK

Try 2 MC 10×33.6 Grade 50 steel sections.

Check shear capacity:

Maximum shear force, $V_{max} = T_{max} * 6 = 16512*6 = 99072$ lb = 99.1 kips

Required area, $A = 99.1/16.5 = 6.0$ in.^{2}

Shear area provided by 2 MC 10×33.6,

$= 2*d*t_w = 2*10*0.575 = 11.5$ in.^{2} >6.0 in.^{2} OK

where d and t_w are the web depth and width of MC 10×33.6.

Use 2 MC 10 × 33.6 Grade 50 sections.

2.2.7 Anchor lengths

As for the "safety with economy" design, for constructibility, the upper three tiers of ground anchors will be inclined downward at an angle of 20 deg and the lower tier inclined downward at an angle of 15 deg (see Figure 2.2).

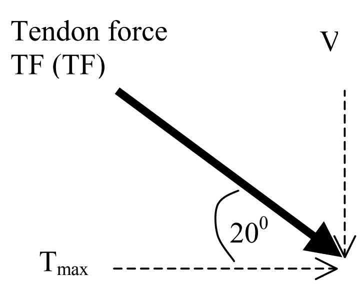
2.2.7.1 Unbonded anchor length, L_i. Using the unbonded length requirements of Figure 8.5 of Strom and Ebeling (2001), the minimum unbonded length for each anchor tier can be determined. These calculations are identical to those provided in Section 2.1.8.1 and are not repeated here. However, the unbonded length determined in Section 2.1.8.1 should be verified using the internal stability analyses procedures described in Strom and Ebeling (2002b). The verification process uses limiting equilibrium procedures, which can be performed by simple hand calculations or GPSS procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to meet internal stability performance requirements (i.e., factor of safety of 1.5 for a stringent displacement control design).

2.2.7.2 Bonded length of anchors, L_b. The usual practice is for the wall designer to specify the anchor capacity and any right-of-way and easement constraints required of the anchorage system. It is up to the tieback anchor contractor, usually a specialty subcontractor, to propose the type of anchorage system to be used to meet the wall design requirements. A preliminary estimate of the bond length (L_b) required to develop the ground anchors for the stringent displacement control design is provided below.

The horizontal anchor forces T_2 and T_3 are all of equal magnitude and correspond to maximum horizontal anchor force T_{max} (Section 2.2.5). Because the horizontal anchor forces T_1 and T_4 are within 7 percent and 2 percent, respectively, of this T_{max} value, the bond length computations will be made using the tendon force value of T_{max}. The computed bond length will be slightly conservative for anchor tendon 1.

With 6-ft spacing between soldier beams and using T_{max} = 16,512 lb/ft, for all anchors, the maximum anchor (tendon) force TF is

$$TF = \frac{T_{max} * 6}{\cos 20°} = \frac{16512 * 6}{\cos 20°} = 105430 \text{ lb} \approx 105.4 \text{ kips}$$



The empirical method used in the following computations for bond length of anchors is for preliminary design purposes. It is up to the tieback anchor contractor, usually a specialty subcontractor, to propose the type of anchorage system to be used to meet the wall design requirements. The final (working load) anchor capacity shall be verified by proof-testing each anchor in the field and performance-testing select anchors (Strom and Ebeling 2002b).

It is assumed in the following computations that a strait shaft pressure-treated ground anchor will be used. The interrelationship between the maximum anchor (tendon) force TF and the ultimate anchor capacity TF_{ult} is given by

$$TF = \frac{TF_{ult}}{FS}$$

The factor of safety against anchor failure is set equal to 2.0 for tieback walls. Recall that in this design problem, TF is equal to 105.4 kips. By this equation the minimum value of the ultimate tieback anchor capacity TF_{ult} is equal to 210.8 kips.

Rearranging Equation 1.1, the minimum length of anchor bond zone length L_b is given by

$$L_b = \frac{TF_{ult}}{RLT_{ult}}$$

where RLT_{ult} is the ultimate capacity of rate of load transfer (kips per foot).

Using the data contained in Figure 23 in Andersen (1984) the ultimate load-transfer rate RLT_{ult} for loose sand is set equal to 6 kips per lineal ft. The minimum value for L_b is

$$L_b = \frac{210.8}{6} = 35.1 \text{ ft}$$

The computed minimum anchor bond length value is less than 40 ft so the tieback anchorage system is feasible. (Alternatively, a post-grouted (regroutable) ground anchor system may be considered since it is likely to result in a lower L_b value.)

2.2.7.3 Total anchor lengths ($Lt_i = L_i + L_b$).

Top-tier anchor:

$$Lt_1 = 31.8 + 35.1 = 66.9 \text{ ft} \approx 67 \text{ ft}$$

Second-tier anchor:

$$Lt_2 = 26.35 + 35.1 = 61.45 \text{ ft} \approx 62 \text{ ft}$$

Third-tier anchor:

$$Lt_3 = 20.9 + 35.1 = 56 \text{ ft}$$

Lower-tier anchor:

$$Lt_4 = 15.56 + 35.1 = 50.66 \text{ ft} \approx 51 \text{ ft}$$

The total anchor length (bonded + unbonded length) determined above should be verified using the external stability analysis procedures described in Strom and Ebeling (2002b). This verification process also uses limiting equilibrium procedures, which can be performed by simple hand calculations or GPSS procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to prevent ground mass stability failure (i.e., to meet external stability performance requirements with a factor of safety of 1.5 for a stringent displacement control design).

2.2.8 Anchor strands

The number of 0.6-in.-diam ASTM A416, Grade 270, strands (ASTM 1999) required to meet stringent displacement control design requirements is determined. It is assumed that the final design force after losses will be based on an allowable anchor stress of 0.6 f_y, or 35.2 kips per strand.

Use the same maximum anchor load TF = 105.4 kips for sizing all four anchor strands (since $T_2 = T_3 = T_{max}$, and T_1 and T_4 are within 7 percent and 2 percent, respectively, of T_{max}).

From Table 8.5, Strom and Ebeling (2001)

Capacity of three 0.6-in. strands = 105.6 kips > 105.4 kips OK

Use three 0.6-in. strands.

2.2.9 Drill-in shaft diameter

The drilled-in soldier beam will be fabricated from a pair of MC 10×33.6 shapes (using Grade 50 steel), as discussed in Section 2.2.6. Additionally, a 12-in.-diameter hole, with casing, will be used to construct the anchor bond zone for all anchors, as discussed in Section 2.2.7.2.

The depth, d, and flange width, b_f, of an MC 10×33.6 are

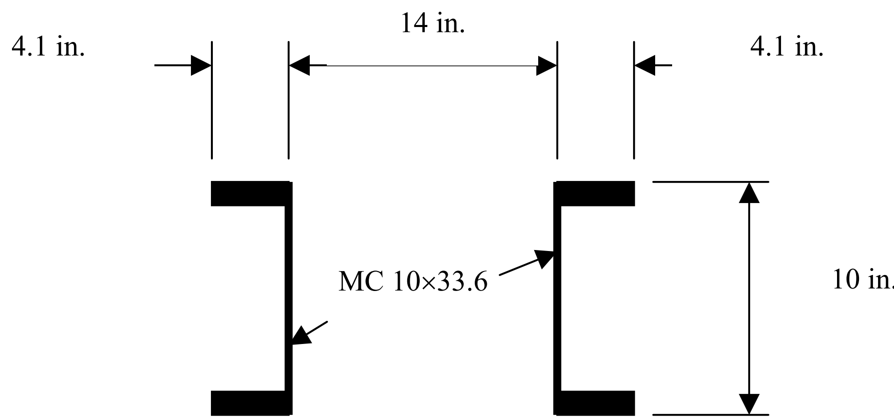
d = 10 in.

and

b_f = 4.1 in.

From Table 8.6, Strom and Ebeling (2001), trumpet diameter for three 0.6-in. strands and Case I corrosion protection = 5-7/8 in.

The distance between channels is set equal to 14 in. to allow ample room for a casing to keep the hole open in the loose sand until the anchor zone grout has been placed. For anchor zone details see Figure 10.2(b), Strom and Ebeling (2001).



The diameter for the drilled shaft (b) required to install the soldier beams is determined next.

For the previously described configuration of the pair of MC 10×33.6 shapes, the diagonal (from flange tip to flange tip) is given by

$$diagonal = \sqrt{d^2 + (2b_f + clear\ spacing)^2}$$

$$diagonal = \sqrt{(10)^2 + (2 \bullet 4.1 + 14)^2}$$

$$diagonal = \sqrt{(10)^2 + (22.2)^2}$$

$$diagonal = 24.35 \text{ in.}$$

To install the fabricated pair of MC 10×33.6 shapes, the diameter of the drilled shaft (b) must be greater than the flange tip to flange tip diagonal of 24.35 in. Use 26-in.-diameter drilled shaft.

2.2.10 Temporary timber lagging

Lagging selection for the stringent displacement control design is identical to that indicated for the "safety with economy" design (see Section 2.1.11).

2.2.11 Soldier beam toe embedment

As with the "safety with economy" design, soldier beam toe embedment requirements for both vertical and horizontal loads must be determined. For the stringent displacement control design with respect to the ***vertical component*** of prestressed anchor load:

$$
\begin{aligned}
\sum V &= \left(V_1 + V_2 + V_3 + V_4\right) \\
&= \left[\left(T_1 + T_2 + T_3\right) * \tan 20° + T_4 * \tan 15°\right] * 6 \\
&= \left[\left(15424 + 2 * 16512\right) * \tan 20° + 16168 * \tan 15°\right] * 6 \\
&= 131795 \text{ lb} = 131.8 \text{ kips}
\end{aligned}
$$

FHWA-SA-99-015 (page 95) general design recommendations for concrete backfill of predrilled holes include the use of structural concrete from the bottom of the hole to the excavation base and a lean-mix concrete for the remainder of the hole. The design concept is to provide maximum strength and load transfer in the permanently embedded portion of the soldier beam while providing a weak concrete fill in the upper portion, which can be easily removed and shaped to allow lagging installation. However, contractors often propose the use of lean-mix concrete backfill for the full depth of the hole to avoid the delays associated with providing two types of concrete in relatively small quantities. This design example follows the granular soil design examples given in Section 10.1 of FHWA-RD-97-130 and in Example 1 of Appendix A of FHWA-SA-99-015, assuming that lean-mix concrete is used for the full depth of the hole.

The following computations are made to determine total force that the drilled-in shaft foundation must resist. A 16-ft depth of penetration is assumed in these computations for a 50-ft exposed wall height.

The total drilled-in soldier beam shaft weight assuming a 16-ft toe length is equal to the vertical component of anchor force plus the weight of a pair of MC 10×33.6 plus the weight of lean-mix concrete backfill plus the weight of timber lagging. The magnitude of each of these forces is summarized in the following five steps:

a. Vertical component of anchor force = 131.8 kips

b. Weight of 2 MC 10×33.6 channels for 66-ft length = 2*0.0336*66 = 4.44 kips

c. Computation of the weight of lean-mix concrete backfill for a drilled-in soldier beam of length 66 ft:

(1) Weight of lean-mix concrete backfill for a drilled shaft diameter (d_s) of 26 in. and a drilled-in soldier beam cylinder of length 66 ft:

$$\text{Total area} = \pi \bullet \frac{d_s^{\,2}}{4}$$

$$\text{Total area} = \pi \bullet \frac{(26)^2}{4}$$

$$\text{Total area} = 530.93\ \text{in.}^2 = 3.687\ \text{ft}^2$$

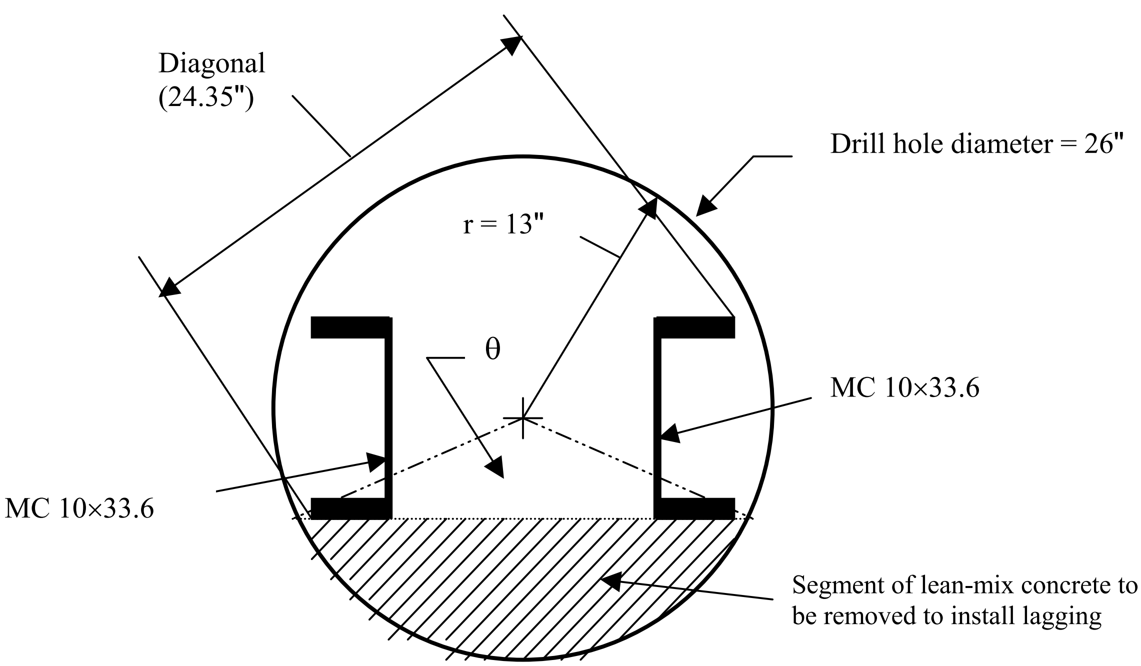
$$\text{Gross weight} = \left(0.145\frac{\text{kips}}{\text{ft}^3}\right) \bullet \text{Total area} \bullet 66\ \text{ft}$$

$$\text{Gross weight} = \left(0.145\frac{\text{kips}}{\text{ft}^3}\right) \bullet (3.687) \bullet 66$$

$$\text{Gross weight} = 35.28\ \text{kips}$$

That is, the gross weight of a 66-ft-high, 26-in.-diameter lean-mix concrete cylinder is 35.28 kips. (This does not account for the weight of lean-mix concrete removed when placing the lagging.)

(2) Reduction in gross weight of a 66-ft-long cylinder for removal of the lean-mix concrete backfill during lagging installation. Compute the area of lean-mix concrete to be removed down the 50 ft (exposed) of height in front of the flanges of the pair of MC 10×33.6 shapes:



Computation of the segment (of circle) area in front of flanges to be removed:

$$\theta = 2 \bullet \left[\cos^{-1}\left(\frac{\text{half channel depth}}{\text{radius}} \right) \right] = 2 \bullet \left[\cos^{-1}\left(\frac{5}{13} \right) \right]$$

$$= 134.76 \text{ deg}$$

where

$$\text{half channel depth} = \frac{d}{2} = \frac{10}{2} = 5 \text{ in.}$$

$$r = \text{radius} = \frac{\text{diameter}}{2} = \frac{d_s}{2} = \frac{26}{2} = 13 \text{ in.}$$

$$\text{Segment area} = \pi r^2 \frac{\theta}{360} - r^2 \frac{\sin \theta}{2} = 138.75 \text{ in.}^2 = 0.96 \text{ ft}^2$$

The exposed wall height equals 50 ft. The area of lean-mix concrete to be removed down the 50 ft of exposed wall in front of the flanges for the pair of MC 10×33.6 shapes is equal to 0.963 ft^2 per ft of exposed wall height. For this 26-in.-diameter drilled-in shaft, the area removed represents approximately 26 percent of the cross-sectional area of the original 26-in.-diameter (cylinder) of lean-mix concrete per ft of exposed height.

Computation of the weight of lean-mix concrete removed during placement of lagging over the 50 ft of exposed height of wall:

$$\text{Weight removed} = \left(0.145 \frac{\text{kips}}{\text{ft}^3} \right) \bullet \text{Segment area} \bullet H$$

$$\text{Weight removed} = \left(0.145 \frac{\text{kips}}{\text{ft}^3} \right) \bullet 0.963 \text{ ft}^2 \bullet 50 \text{ ft} = 6.98 \text{ kips}$$

(3) Computation of the weight of lean-mix concrete backfill for a drilled-in soldier beam of length 66 ft less the weight removed during placement of lagging:

Lean-mix net weight = Gross weight - weight removed

Lean-mix net weight = 35.28 - 6.98 = 28.3 kips

d. Computation of the weight of timber lagging over 50-ft exposed height for a span of 6 ft:

$$\text{Lagging weight} = \left(0.05\frac{\text{kips}}{\text{ft}^3}\right) \bullet \text{span} \bullet \text{Height} \bullet \text{thickness}$$

$$\text{Lagging weight} = \left(0.05\frac{\text{kips}}{\text{ft}^3}\right) \bullet 6\text{ ft} \bullet 50\text{ ft} \bullet \left(\frac{3}{12}\right) = 3.75\text{ kips}$$

e. Computation of the applied total drilled-in soldier beam axial load:

$$\text{Q}_{applied} = \sum V + \text{Weight of channels} + \text{Lean-mix net weight} + \text{Lagging weight}$$

$$\text{Q}_{applied} = 131.8\text{ kips} + 4.44\text{ kips} + 28.3\text{ kips} + 3.75\text{ kips} = 168.29\text{ kips}$$

Thus, for the 26-in.-diameter drilled-in shaft with a 16-ft depth of penetration, the applied axial load is equal to 168.29 kips.

2.2.12 Depth of toe penetration, D

This subsection outlines the depth of penetration computations. For a drilled-in shaft,

Ultimate axial resistance = skin friction resistance + tip resistance

Hence:

$$Q_{ult} = Q_{skin} + Q_{tip} \qquad \text{(see Equation 8.25 in Strom and Ebeling 2001)}$$

The factors of safety for axial capacity of drilled-in soldier beams in granular soil are

$FS_{skin} = 2.0$ and $FS_{skin} = 2.5$

according to Table 8.9 in Ebeling and Strom (2001) and Table 8.9 in FHWA-SA-99-015. Thus, the allowable axial load Q_{all} is given by

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} \qquad \text{(modified form of Equation 8.18 in Strom and Ebeling 2001)}$$

The traditional potential foundation failure mode due to axial loading is based on the assumption of the drilled-in shaft being fully effective in transferring the applied vertical load from the pair of steel channels through the lean-mix concrete mix to the surrounding soil. The corresponding traditional computation assumes the axial capacity is due to the drilled-in shaft acting as a single structural unit within the surrounding granular soil media. FHWA-SA-99-015 (page 95) and FHWA-RD-97-130 (page 90) note that for lean-mix backfilled drilled-in shafts, the lean-mix concrete may not be sufficiently strong to allow vertical load transfer from the soldier beam to the concrete. Consequently, a second potential failure mode must also be considered: The alternative potential failure mode assumes the soldier beam "punches" through the lean-mix, in which case the drilled-in shaft cross-section (assumed to be rectangular) will not be effective in transferring the load to the surrounding soil. Both potential failure modes are evaluated and the smallest capacity is used in the design. These computations are demonstrated in the following two sets of analyses. Note that in each set of computations a different value for depth of penetration D was used. Since the mechanisms for the two potential modes of failures are different, the minimum depth of penetration values required to satisfy the factors of safety against failure will also differ.

2.2.12.1 Analysis 1: Drilled-in shaft capacity (with shaft acting as single unit). In Section 2.2.9 the diameter of the drilled-in shaft was established to be 26 in. By trial and error using the following design analysis procedure it is determined that a depth of penetration (D) equal to 16 ft is required to meet the aforementioned factor of safety requirements. The soldier beam length is equal to 66 ft (= H + D = 50 ft + 16 ft).

a. *Ultimate skin friction*. The ultimate resistance due to skin friction, Q_{skin}, is given by

$$Q_{\text{skin}} = f_{skin} \bullet A_{cylinder}$$

The average unit skin friction is computed using the Strom and Ebeling (2001) Equation 8.26 to be

$$f_{skin} = \beta \bullet \sigma'_{ave} \quad \text{with the limitation that } f_{skin} < 4 \text{ ksf}$$

$$\beta = 1.5 - 0.135 \bullet \left[\frac{H+D}{2}\right]^{0.5} \text{ with the limitation that } 0.25 < \beta < 1.25$$

$$\beta = 1.5 - 0.135 \bullet \left[\frac{50+16}{2}\right]^{0.5} = 0.724$$

FHWA-HI-88-042 (1988) stated that β is independent of soil strength because drilling disturbance reduces the friction angle to a common value regardless of initial soil strength.

$$\sigma'_{ave} = \gamma \bullet \left(\frac{H+D}{2}\right) = 115 \bullet \left(\frac{50+16}{2}\right) = 3795 \text{ psf}$$

resulting in

$$f_{skin} = 0.724 \bullet 3795 \text{ psf} \bullet \left(\frac{\text{kips}}{1000 \text{ lbf}} \right) = 2.75 \text{ ksf}$$

The surface area of the drilled-in shaft is given by

$$A_{cylinder} = \pi \bullet (diameter) \bullet D$$

$$A_{cylinder} = \pi \bullet \left[26 \text{ in.} \bullet \left(\frac{\text{ft}}{12 \text{ in.}} \right) \right] \bullet 16 \text{ ft} = 108.91 \text{ ft}^2$$

Thus, the ultimate resistance due to skin friction along the 14-ft-long depth of penetration of the drilled-in shaft is

$$Q_{skin} = f_{skin} \bullet A_{cylinder} = 2.75 \bullet 108.91 = 299.5 \text{ kips}$$

b. *Ultimate tip resistance.* The ultimate tip resistance due to end bearing, Q_{tip}, is given by

$$Q_{tip} = q_b \bullet A_{tip}$$

The unit end bearing ultimate resistance is computed using the Strom and Ebeling (2001) Table 8.10 relationship:

$q_b = 1.2 \bullet$ (uncorrected SPT N-value) in units of ksf and with the limitation that the uncorrected SPT N-value be less than 75.

$$q_b = 1.2 \bullet (15) = 18 \text{ ksf}$$

The cross-sectional area of the tip is

$$A_{tip} = \pi \bullet \frac{(\text{diameter})^2}{4}$$

$$A_{tip} = \pi \bullet \frac{\left[26 \text{ in.} \bullet \left(\frac{\text{ft}}{12 \text{ in.}} \right) \right]^2}{4} = 3.687 \text{ ft}^2$$

Thus, the ultimate tip resistance of the 26-in.-diameter drilled-in shaft is

$$Q_{tip} = q_b \bullet A_{tip} = 18 \bullet 3.687 = 66.4 \text{ ksf}$$

c. Ultimate axial load resistance. The ultimate axial load Q_{ult} is computed to be

$$Q_{ult} = Q_{skin} + Q_{tip} = 299.5 + 66.4 = 365.9 \text{ kips}$$

Note that skin friction provides 82 percent of the ultimate axial load resistance, while tip resistance due to end bearing provides 18 percent of this ultimate axial load value.

d. Allowable axial load. The allowable axial load Q_{all} is computed to be

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} = \frac{299.5}{2.0} + \frac{66.4}{2.5} = 149.75 + 26.56 = 176.31 \text{ kips}$$

Note that skin friction provides 85 percent of the allowable axial load resistance, while tip resistance due to end bearing provides 15 percent of this allowable axial load value.

The allowable axial load Q_{all} for this 26-in.-diameter drilled-in shaft with an assumed 16-ft depth of embedment is 176.31 kips, which is 8.02 kips larger than the applied axial load of 168.29 kips (see Section 2.2.11), i.e., $Q_{applied} > Q_{all}$. Thus, a 16-ft depth of penetration is acceptable for this assumed potential mode of foundation failure. Recall that this potential mode of foundation failure assumes the shaft acting as single unit.

Solution process: The procedure used to determine the depth of penetration D in this problem was as follows: assume a depth of penetration; compute the ultimate axial load resistance Q_{ult}; compute the allowable axial load Q_{all}; compute the total applied axial load for the drilled-in soldier beam system $Q_{applied}$ (Section 2.2.11); adjust the depth of penetration D as necessary and repeat computations until Q_{all} is approximately equal to $Q_{applied}$. Ensure that for the final value of D used in the computations Q_{all} is greater than or equal to $Q_{applied}$.

2.2.12.2 Analysis 2: "Punching" soldier beams capacity. The drilled-in soldier beam backfilled with a lean-mix concrete has the potential not to act as a single structural unit in which the axial load is transferred from the pair of channels through the lean-mix to the surrounding soil foundation but where the pair of channels "punches" through the lean-mix concrete backfill. In Section 2.2.9 the diameter of the drilled-in shaft was established to be 26 in. By trial and error using the following design analysis procedure it is determined that a depth of penetration (D) equal to 11 ft is required to meet established factor of safety requirements. (Note that this value of D differs from the 16-ft value used in Analysis 1 computations. The authors of this report are demonstrating that the minimum required depth of penetration is not the same for the two different types of failure modes.) For the Analysis 2 procedure the soldier beam length is equal to 61 ft (= H + D = 50 ft + 11 ft). As an alternative the designer could verify the depth of penetration established by the design Analysis 1 procedure satisfies factor of safety requirements for the Analysis 2 procedure.

The following computations assume the pair of soldier beam channels will punch through the lean-mix concrete backfill rather than transfer the load through the backfill to the ground. Note that in these computations the diameter of the drilled shaft is not used (i.e., the drilled shaft cross-section will not be effective in transferring the load to the surrounding soil). Instead, the rectangular "box" perimeter of the pair of channels is used in both the skin friction and tip resistance computations.

a. *Ultimate skin friction.* The ultimate resistance due to skin friction, Q_{skin}, is given by

$$Q_{skin} = f_{skin} \bullet A_{box}$$

The average unit skin friction for this "punching" mode of failure is computed using the Strom and Ebeling (2001) Equation 8.20 to be

$$f_{skin} = K \bullet \sigma'_{ave} \bullet \tan(\delta)$$

with

$$\sigma'_{ave} = \gamma \bullet \left(\frac{H+D}{2}\right) = 115 \bullet \left(\frac{50+11}{2}\right) = 3508 \text{ psf}$$

FHWA-RD-97-130 (page 94) and FHWA-SA-99-015 (page 95) note that when a lean-mix concrete backfill is used in a drilled-in shaft, f_{skin} is computed using K = 2 and δ = 35 degrees in the f_{skin} equation (see page 180 in Strom and Ebeling 2001). Note that these parameter values are specific to the "punching" mode of failure through the lean-mix concrete. Thus, f_{skin} becomes

$$f_{skin} = 2 \bullet \left[3508 \text{ psf} \bullet \left(\frac{\text{kips}}{1000 \text{ lb}}\right)\right] \bullet \tan(35) = 4.913 \text{ ksf}$$

The surface area of the rectangular "box" defined by the perimeter of the pair of channels is given by

$$A_{box} = [2 \bullet (\text{channel depth}) + 2 \bullet (\text{flange-to-flange width})] \bullet D$$

$$A_{box} = \left[2 \bullet (\text{channel depth}) + 2 \bullet \begin{pmatrix} 2 \bullet b_f + \text{clear space} \\ \text{between channels} \end{pmatrix}\right] \bullet D$$

$$A_{box} = [2 \bullet (10 \text{ in.}) + 2 \bullet (2 \bullet 4.1 \text{ in.} + 14 \text{ in.})] \bullet \left(\frac{1}{12}\right) \bullet 11 \text{ ft}$$

$$= [(20 \text{ in.}) + (44.4 \text{ in.})] \bullet \left(\frac{1}{12}\right) \bullet 11 \text{ ft} = 59.03 \text{ ft}^2$$

Thus, the ultimate resistance due to skin friction along the 10-ft-long depth of penetration of the drilled-in shaft is

$$Q_{skin} = f_{skin} \bullet A_{box} = 4.913 \text{ ksf} \bullet 59.03 = 290 \text{ kips}$$

b. *Ultimate tip resistance*. The ultimate tip resistance due to end bearing, Q_{tip}, is given by

$$Q_{tip} = q_b \bullet A_{tip}$$

The unit end bearing ultimate resistance is computed using the Strom and Ebeling (2001) Equation 8.21 relationship

$$q_b = \sigma'_v \bullet N_q$$

where

$$\sigma'_v = \gamma \bullet D$$

$$\sigma'_v = 115 \bullet 11 = 1265 \text{ psf}$$

According to FHWA-RD-97-130 (page 94), a value of N_q in the middle range recommended by Meyerhoff gives the best estimate of the end bearing capacities. Using Figure 8.11 in Strom and Ebeling (2001), this midrange Meyerhoff N_q value is equal to 40 for ϕ equal to 30 degrees. This results in a unit end bearing ultimate resistance equal to

$$q_b = 1265 \bullet \left(\frac{1}{1000}\right) \bullet 40 = 50.6 \text{ ksf}$$

The cross-sectional area of the rectangular "box" tip is

$$A_{tip} = (\text{channel depth}) \bullet (\text{flange - to - flange width})$$

$$A_{tip} = (\text{channel depth}) \bullet (2 \bullet b_f + \text{clear space between channels})$$

$$A_{tip} = [(10 \text{ in.}) \bullet (2 \bullet 4.1 \text{ in.} + 14 \text{ in.})] = [(10 \text{ in.}) \bullet (22.2 \text{ in.})] = 222 \text{ in.}^2$$

Thus, the ultimate tip resistance of the 26-in.-diameter drilled-in shaft is

$$Q_{tip} = q_b \bullet A_{tip} = 50.6 \bullet 222 \bullet \left(\frac{1}{144}\right) = 78 \text{ ksf}$$

c. Ultimate axial load resistance. The ultimate axial load Q_{ult} is computed to be

$$Q_{ult} = Q_{skin} + Q_{tip} = 290 + 78 = 368 \text{ kips}$$

Note that skin friction provides 79 percent of the ultimate axial load resistance, while tip resistance due to end bearing provides 21 percent of this ultimate axial load value.

d. Allowable axial load. The allowable axial load Q_{all} is computed to be

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} = \frac{290}{2.0} + \frac{78}{2.5} = 145 + 31.2 = 176.2 \text{ kips}$$

Note that skin friction provides 82 percent of the allowable axial load resistance, while tip resistance due to end bearing provides 18 percent of this allowable axial load value.

The allowable axial load Q_{all} for this 26-in.-diameter drilled-in shaft with an assumed 10-ft depth of embedment is 176.2 kips, which is 10.9 kips larger than the applied axial load of 165.3 kips (computations not shown but follow those made in Section 2.2.11 using a 11-ft depth of penetration). Thus, a 11-ft depth of penetration is acceptable for this assumed potential mode of foundation failure. Recall that this potential mode of foundation failure assumes the soldier beam "punches" through the lean-mix concrete backfill.

Solution process: The procedure used to determine the depth of penetration D in this problem was as follows: assume a depth of penetration; compute the ultimate axial load resistance Q_{ult}; compute the allowable axial load Q_{all}; compute the total applied axial load for the drilled-in soldier beam system $Q_{applied}$ (following the procedure outlined in Section 2.2.11); adjust the depth of penetration D as necessary and repeat computations until Q_{all} is approximately equal to $Q_{applied}$. Ensure that for the final value of D used in the computations Q_{all} is greater than or equal to $Q_{applied}$.

2.2.12.3 Concluding remarks. The minimum required depths of penetration were computed in this section for two potential failure modes. It was found in design Analysis 1 that a 16-ft minimum depth of penetration is required to be safe by the traditional potential foundation failure mode in which the drilled-in shaft acts as a single structural unit within the surrounding granular soil media. It was found in design Analysis 2 that an 11-ft minimum depth of penetration is required for the system to be safe against the alternative potential failure mode that assumes the soldier beam "punches" through the lean-mix. Therefore, the required depth of penetration D for this drilled-in shaft retaining structure is 16 ft.

2.2.13 Lateral capacity of soldier beam toe

Assume, based on vertical load requirements that the final toe penetration (D) is 16 ft.

Check lateral capacity of soldier beam toe:

Subgrade reaction per foot of wall, R = 3,096 lb/ft (Section 2.2.4)

Total toe reaction = 3,096*6= 18,576 lb = 18.6 kips

A spreadsheet incorporating the Wang-Reese passive resistance equations (Table 2.4) is used to determine lateral resistance of the soldier beam toe following the procedure outlined in Section 8.7 of Strom and Ebeling (2001) or Section 6.2 in FHWA-RD-97-130. Note that the soldier beam width of 1.85 ft (22.2 in.) is used for drilled shafts backfilled with lean concrete as per FHWA-RD-97-130 (page 111) recommendations. If structural concrete is used to backfill the shaft, then the drilled shaft diameter (= 26 in.) would be used in the computations. Alpha and beta in this table are angles used to define the three-dimensional geometrical configuration of the "passive" failure wedge developing in front of the soldier beam on the excavated side (refer to Figure 8.6 in Strom and Ebeling 2001). The Wang-Reese definitions are

$$\beta = 45 + \frac{\phi'}{2} = 45 + \frac{30}{2} = 60\,\text{deg}$$

and

$$\alpha = \frac{\phi'}{3} \text{ to } \frac{\phi'}{2}\,\text{for loose sands, and}\ \phi\ \text{ for dense sands}$$

$$\alpha = \frac{30}{3} \text{ to } \frac{30}{2} = 10 \text{ to } 15\,\text{deg, use}\,10\,\text{deg}$$

Passive resistance back-calculated for soldier beam and lagging systems compares favorably with passive resistance equations developed by Wang and Reese (1986). Several passive failure mechanisms must be evaluated for each increment of soldier beam embedment and the pressure associated with the governing failure condition summed over each increment of depth to determine the soldier beam total passive resistance. This process is summarized for the cohesionless soil example in Table 2.4 for the stringent displacement control. In Table 2.4, the pressures attributed to the various failure mechanisms are provided in columns 5 through 8, and the pressures associated with the governing failure condition are indicated in column 9. The equation numbers referenced in the various columns of the table refer to equations from FHWA-RD-97-130. Similar equations can be found in FHWA-SA-99-015 and in Strom and Ebeling (2001). (Table 2.2 provides the reference equation number associated with each of these three references.)

Table 2.4
Spreadsheet for Computing Passive Resistance for Sand (“Stringent Displacement Control” Design)

INPUT VARIABLES

unit weight(kip/ft^3)	wall height(ft)	beam width(ft)	beam spacing(ft)	phi	alpha	beta	toe depth(ft)	toe reaction(kip)
0.115	50	1.85	6	30	10	60	16	18.6

COMPUTED FUNCTIONS

tangent phi	tan beta	tan (beta-phi)	tan alpha	sin beta	cos alpha	sine phi	cos phi	cos beta	Ko	Ka	Kp	Sc
0.577	1.7321	0.5774	0.1763	0.8660	0.9848	0.5000	0.8660	0.5000	0.5000	0.3329	2.9927	6

SPREADSHEET FOR EVALUATING TOE CAPACITY FOR SAND(“stringent displacement contro” design)

Toe depth (D)	*wing width*	(Eq 6.14) *interaction height, di*	(Eq 6.13) *wedge resistance (kip/ft)*	*wedge resistence D=di (kip/ft)*	*wedge resistance D=di, a=0 (kip/ft)*	(Eq 6.15) *intersecting wedge resistance (kip/ft)*	(Eq 6.16) *flow resistance (kip/ft)*	*critical resistance (kip/ft)*	(Eq 6.17) *Rankine passive resistance (kip/ft)*	*passive resistance (kip/ft)*	*total passive force (kip)*	*total active force (kip)*	*net passive force (kip)*	*factor of safety*
col.1	*col 2*	*col 3*	*col 4*	*col 5*	*col 6*	*col 7*	*col 8*	*col 9*	*col 10*	*col 11*	*col 12*	*col 13*	*col 14*	*col 15*
0	0.0000	-9.8229	0.0000	13.0727	6.9287	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.9143	-1.9143	-0.1029
1	0.3054	-8.8229	0.7057	10.0816	5.1249	0.7057	4.9508	0.7057	2.5812	0.7057	0.3528	1.9335	-1.5806	-0.0850
2	0.6108	-7.8229	1.7877	7.4670	3.5702	1.7877	9.9016	1.7877	5.1623	1.7877	1.5995	3.9052	-2.3057	-0.1240
3	0.9162	-6.8229	3.2460	5.2286	2.2644	3.2460	14.8524	3.2460	7.7435	3.2460	4.1163	5.9152	-1.7989	-0.0967
4	1.2216	-5.8229	5.0807	3.3666	1.2076	5.0807	19.8032	5.0807	10.3247	5.0807	8.2797	7.9636	0.3161	0.0170
5	1.5270	-4.8229	7.2917	1.8809	0.3998	7.2917	24.7539	7.2917	12.9058	7.2917	14.4659	10.0502	4.4157	0.2374
6	1.8325	-3.8229	9.8790	0.7716	-0.1590	9.8790	29.7047	9.8790	15.4870	9.8790	23.0512	12.1751	10.8762	0.5847
7	2.1379	-2.8229	12.8427	0.0386	-0.4688	12.8427	34.6555	12.8427	18.0682	12.8427	34.4121	14.3382	20.0738	1.0792
8	2.4433	-1.8229	16.1827	-0.3181	-0.5297	16.1827	39.6063	16.1827	20.6493	16.1827	48.9248	16.5397	32.3851	1.7411
9	2.7487	-0.8229	19.8990	-0.2984	-0.3415	19.8990	44.5571	19.8990	23.2305	19.8990	66.9657	18.7795	48.1862	2.5907
10	3.0541	0.1771	23.9917	0.0976	0.0956	23.9897	49.5079	23.9897	25.8117	23.9897	88.9101	21.0575	67.8526	3.6480
11	3.3595	1.1771	28.4607	0.8699	0.7817	28.3725	54.4587	28.3725	28.3928	28.3725	115.0912	23.3738	91.7173	4.9310
12	3.6649	2.1771	33.3061	2.0186	1.7167	33.0043	59.4095	33.0043	30.9740	30.9740	144.7644	25.7284	119.0360	6.3998
13	3.9703	3.1771	38.5278	3.5436	2.9008	37.8850	64.3603	37.8850	33.5551	33.5551	177.0290	28.1213	148.9076	8.0058
14	4.2757	4.1771	44.1258	5.4449	4.3339	43.0147	69.3110	43.0147	36.1363	36.1363	211.8747	30.5525	181.3222	9.7485
15	4.5811	5.1771	50.1001	7.7226	6.0159	48.3934	74.2618	48.3934	38.7175	38.7175	249.3016	33.0220	216.2796	11.6279
16	**4.8866**	**6.1771**	**56.4508**	**10.3766**	**7.9469**	**54.0211**	**79.2126**	**54.0211**	**41.2986**	**41.2986**	**289.3097**	**35.5297**	**253.7799**	**13.6441**

The computations summarized in Table 2.4 are for the 50-ft-high tieback wall in granular soil (sand) for a "stringent displacement control" design. These computations explicitly follow those given in the Figure 98 spreadsheet procedure (FHWA-RD-97-130, page 192 for a two-tier, drilled-in soldier beam wall). It should be noted that the FHWA "Granular Soil Design Example" is for a 30-ft-high tieback wall (see Section 10.1, page 171, FHWA-RD-97-130). The differences in toe passive resistance (i.e., Table 2.4 herein versus FHWA Figure 98) are due to the soldier beam width (1.85 ft in Table 2.4 versus 1.067 ft in FHWA report), and the soil properties ($\phi = 30^0$, $\gamma = 115$ psf in Table 2.4 versus $\phi = 29^0$, $\gamma = 108$ psf in FHWA report). The total active force and net passive resistance (columns 13 and 14, respectively) are dependent on wall height (30 ft for the FHWA Figure 98 example versus 50 ft for the Table 2.4 example). In accordance with FHWA-RD-97-130, Table 2.4 includes a total active force reduction (column 13) to account for the active soil pressures acting on the toe of the soldier beam. The total net passive force (i.e., toe passive resistance minus toe active soil pressure) is indicated in column 14 of Table 2.4. The factor of safety is indicated in column 15. Table 2.4 indicates that soldier beams spaced at 6 ft on centers with a 16-ft toe penetration will have a lateral resistance of 253.78 kips. This provides a factor of safety of 13.6, which is greater than the minimum of 2.0 required for a stringent displacement control design.

A summary of the results for the stringent displacement control design is provided in Table 2.5.

2.2.14 Failure planes below the bottom of the cut

Since most cohesionless soils exhibit friction angles greater than 30 degrees, the difference between the total load required to stabilize the cut for failure planes that pass through the corner of the cut versus failure planes that pass beneath the bottom of the cut is typically minor according to FHWA-RD-98-065. However as friction angles drop below 30 degrees, the difference becomes significant, with the total load obtained from the evaluation of failure planes that pass beneath the bottom of the cut being greater than from those that pass through the corner of the cut (FHWA-RD-98-065). For loose sands the failure surface may extend below the bottom of the cut, thereby increasing the total load required to stabilize the cut. A GPSSP analysis can evaluate failure planes passing below the bottom of the cut. The Spencer method considers both force and moment equilibrium. Therefore it is often selected for the GPSSP analysis of cohesionless soil sites. The Spencer method can be used to determine the total load the tieback system must carry to meet internal stability factor of safety requirements established for the project. The total load determined from a Spencer method internal stability limiting equilibrium analysis can be redistributed into an apparent pressure diagram. This apparent pressure diagram should be used as a basis for the design if it provides a greater total load than that obtained from a conventional apparent pressure diagram (one that assumes a "bottom corner of the cut" failure plane condition). GPSSP analyses are described in FHWA-RD-98-065 and in Strom and Ebeling (2002b). GPSSP analyses should always be used to verify that the total load required to meet internal stability safety requirements is equal to or less than that used for the original design.

Table 2.5
Summary of Results for Four-Tier, 50-ft Drilled-In Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Retaining Granular Soils–Stringent Displacement Control Design

Parameter		Value
Wall height		50 ft
Soldier beam spacing		6 ft
Soldier beam design moment		108.7 kip-ft
Soldier beam size		2 MC 10×33.6
Soldier beam length		66 ft
Drill shaft diameter		26 in.
Toe reaction		18.6 kips
Top-tier Anchor	H_1	7 ft, 0 in.
	Anchor inclination	20 deg
	Design load	93.5 kips
	Unbonded length	31.8 ft
	Bonded length	35.1 ft
	Total length	67 ft
	Tendon size	three 0.6-in. strands
Second-tier Anchor	H_2	10 ft, 9 in.
	Anchor inclination	20 deg
	Design load	105.4 kips
	Unbonded length	26.35 ft
	Bonded length	35.1 ft
	Total length	62 ft
	Tendon size	three 0.6-in. strands
Third-tier Anchor	H_3	10 ft, 9 in.
	Anchor inclination	20 deg
	Design load	105.4 kips
	Unbonded length	20.9 ft
	Bonded length	35.1 ft
	Total length	56 ft
	Tendon size	three 0.6-in. strands
Lower-tier Anchor	H_4	10 ft, 9 in.
	Anchor inclination	15 deg
	Design load	100.4 kips
	Unbonded length	15.56 ft
	Bonded length	35.1 ft
	Total length	51 ft
	Tendon size	three 0.6-in. strands

As the soil above the failure plane attempts to move out, shear resistance is mobilized in the soldier beams. Shear in the soldier beams provides additional resistance to soil movement. Predicting soldier beam shear requires the consideration of three possible failure modes: (1) shear in the soldier beam; (2) flow of the soil between the soldier beams: and (3) lateral capacity of the soldier beams. These three failure mechanisms are discussed in Section 4.3.3 of FHWA-RD-98-065. The resistance provided by the soldier beams for each of the three possible failure modes can be estimated and included in a GPSSP analysis, provided the GPSSP used is capable of modeling the soldier beams as reinforcement.

The calculated mass stability (i.e., external stability) slip circles for loose sands can be deep and located beyond or at the end of the usual tieback anchorage location. Ground mass stability in loose sands can be improved by extending the length of the tiebacks. The use of GPSSP analyses for determining the required position of the back of the tieback anchor is covered in Chapter 4 of FHWA-RD-98-065 and in Strom and Ebeling (2002b).

3 Simplified Design Procedures for 50-ft-High Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Retaining Cohesive Soil

The two example problems presented in this chapter deal with the application of the design procedures and guidelines for drilled-in soldier beam systems given in Strom and Ebeling (2001), FHWA-RD-97-130, and FHWA-SA-99-015. A 50-ft wall height (horizontal retained soil surface) with homogenous cohesive retained soil is considered. These design computations follow the cohesive soil design example of Section 10.2 in FHWA-RD-97-130. A "safety with economy" design example is given first, followed by a "stringent displacement control" design example.

3.1 Soil Property Summary

This particular wall is founded in stiff clay. A stiff clay site was selected because soft to medium clay soils with stability numbers ($\gamma H/S_u$) greater than 5 are considered to be potentially dangerous and, as such, the use of a soldier beam and lagging system for support is questionable (see Table 12 of FHWA-SA-99-015). The soil properties selected are per the "Cohesive Soil Design Example" of FHWA-RD-97-130 (Step 2, page 204). The undrained shear strength (S_u) was given as 2,400 psf in FHWA-RD-97-130 for this homogeneous soil site. Using Figure 31 of FHWA-RD-97-130, the EPF for the undrained condition was estimated. For the 50-ft-high wall example calculation to be discussed in the following paragraphs, the EPF is equal to 20 psf, for S_u equal to 2,400 psf by this figure. This is for the short-term loading condition.

For clays, both the undrained (short-term) and drained (long-term) conditions must be evaluated. In the FHWA-RD-97-130 cohesive design example no long-term (drained) shear strength value was provided. FHWA-RD-97-130 estimated the drained shear strength for the long-term condition based on an empirical correlation. This same approach is used in the two design examples given in this chapter. This information is repeated in Appendix A of this report. The clay soil has a plasticity index of 19 and an overconsolidation ratio of 3, according to the FHWA problem statement (Step 2, page 204, FHWA-RD-97-130). It can be estimated—as shown in this report (Appendix A, Figure A.4, and also in the FHWA example)—that the drained friction angle for the long-term condition is equal to 36 deg. (Note that no effective cohesion intercept is included in the Appendix A empirical correlation for both normally consolidated and overconsolidated cohesive soils by this correlation. For further explanation regarding this issue, the reader is referred to Appendix A.) As will be shown in the following calculations, the long-term condition governs the EPF value to be used in determining the design prestress anchor forces.

The soil properties used are in accordance with the cohesive soil, from examples given in FHWA-RD-97-130 (page 204):

- Undrained shear strength S_u = 2,400 psf.
- Unit weight, γ = 132 pcf.
- EPF for undrained (short-term) condition = 20 pcf.
- Friction angle for drained (long-term) condition ϕ = 36 deg.

3.2 "Safety with Economy" Design

For the Corps' "safety with economy" design, a limiting equilibrium approach is used with a factor of safety of 1.3 applied to the shear strength of the soil. (The factor of safety for the limiting equilibrium analysis is increased to 1.5 for the stringent displacement control design.) The total earth pressure load (P_{tl}) is determined based on the limiting equilibrium analysis. Limiting equilibrium calculations for the "safety with economy" design are provided below.

$$\phi_{mob} = \tan^{-1}(\tan\phi / FS)$$

Accordingly,

$$\phi_{mob} = \tan^{-1}\left(\frac{\tan\phi}{1.3}\right)$$

$$= \tan^{-1}\left(\frac{\tan 36°}{1.3}\right) = 29.2°$$

$$K_A = \tan^2\left(45^0 - \frac{\phi_{mob}}{2}\right) = 0.344$$

$$P = K_A \gamma \frac{H^2}{2} = 0.344 * 132 * \frac{50^2}{2} = 56760 \text{ lb/ft}$$

$$\text{Effective pressure factor, EPF} = \frac{\text{P}}{\text{H}^2} = \frac{56760}{50^2} = 22.7 \text{ lb/ft}^3$$

This calculation produces an EPF equal to 22.7 pcf for the long-term (drained) condition. Figure 31 in FHWA-RD-97-130 produces an EPF equal to 20 pcf for the short-term (undrained) condition. Use an EPF equal to 22.7 pcf in the construction of the apparent pressure diagram and in all subsequent computations involving the prestress design anchor forces. This design approach follows the steps taken in the FHWA-RD-97-130 cohesive soil design example of Section 10.2.1 (pages 202-213).

3.2.1 Apparent earth pressure

Check clay classification.

For stiff clay condition,

$$S_u \geq \frac{H}{4}(\gamma - 22.857) \qquad \text{(see paragraph 5.3.5, Strom and Ebeling 2001)}$$

i.e.,

$$H = \frac{4S_u}{\gamma - 22.857} = \frac{4 * 2400}{132 - 22.857} = 88 > 50 \text{ ft} \quad \leftarrow \text{indicates stiff clay}$$

Additionally, for stiff clay,

$$\frac{\gamma H}{c} \leq 4.0$$

and,

$$\frac{\gamma H}{c} = \frac{132 * 50}{2400} = 2.75 < 4.0 \qquad \text{(note that } c = S_u\text{)}$$

Use of stiff clay apparent pressure diagram similar in shape to the one recommended for sand is indicated (see Figure 5.4, Strom and Ebeling 2001).

3.2.2 Anchor points

Using the empirical apparent earth pressure envelope of Figure 5.4 (Strom and Ebeling 2001) and Figure 29 (FHWA-RD-97-130), the vertical anchor intervals with four-tier anchoring for approximate balanced moments are determined. Refer to Figure 3.1 and the following calculations:

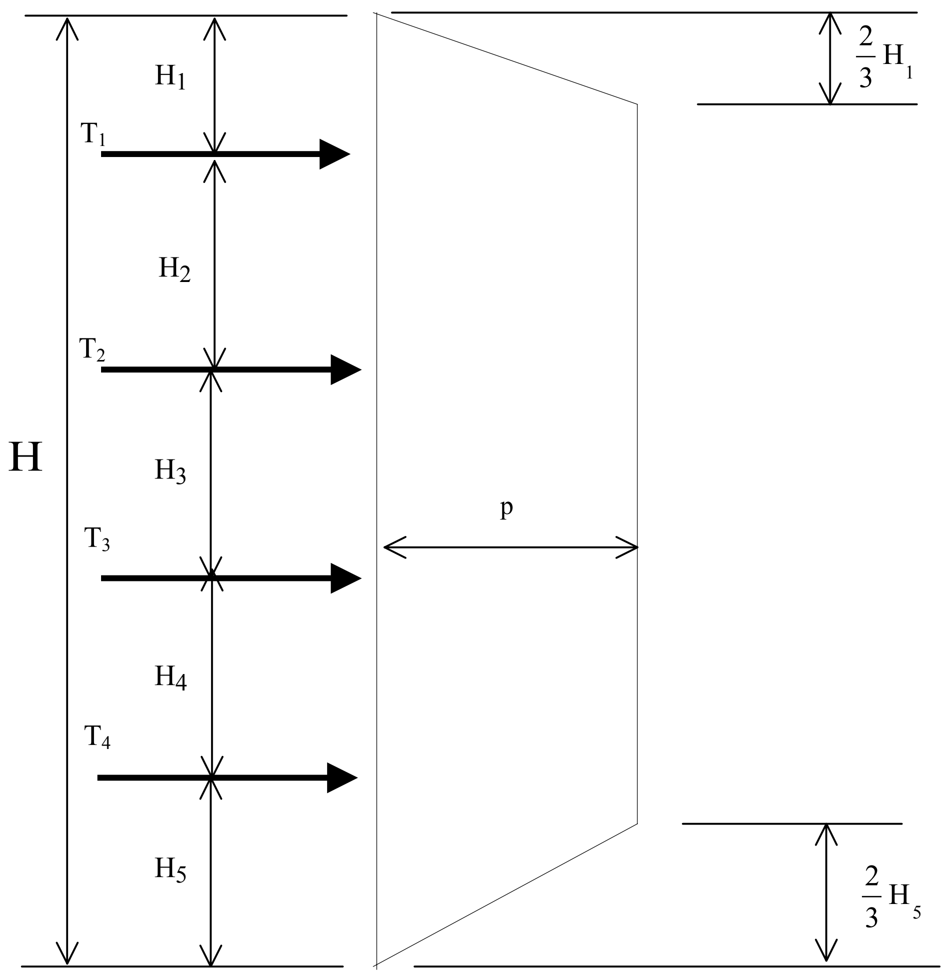


Figure 3.1. Apparent earth pressure

Setting $MM_1 = M_1$ $\quad (_{i=1,2,3\ldots})$

$$\frac{1}{10} H^2_{(2,3,4,5)} \; p = \frac{13}{54} H_1^2 p$$

where

$H_{(2,3,4,5)}$ denotes vertical distances between anchors, i.e., H_2, H_3, H_4, H_5, assumed equal i.e.,

$$H_{(2,3,4,5)} = \sqrt{\frac{130}{54}} H_1 = 1.55 H_1$$

thus, with $H_2 = H_3 = H_4 = H_5$

$$H = H_1 + H_2 + H_3 + H_4 + H_5 = H_1 + 4\ (1.55\ H_1)$$

$$50 = 7.2 H_1$$

therefore,

$$H_1 \approx 6.944 \text{ ft}$$

and

$$H_2 = H_3 = H_4 = H_5 = \frac{H - H_1}{\#\text{ of vertical anchor spacing}}$$

$$= \frac{50 - 6.944}{4} = 10.764 \text{ ft}$$

Try $H_1 = 7'-0"$ and $H_2 = H_3 = H_4 = H_5 = 10'\text{–}9"$

Hence,

$$p = \frac{P_{tl}}{\left(H - \frac{H_1}{3} - \frac{H_5}{3} \right)} \qquad \text{(see Figure 5.4b, Strom and Ebeling 2001)}$$

$$p = \frac{56760}{50 - \frac{7}{3} - \frac{10.75}{3}} = 1288 \text{ psf}$$

3.2.3 Bending moments on soldier beam

Using the information contained in Figure 5.4b (Strom and Ebeling 2001), the cantilever moment (M_1) and the maximum interior span moments (MM_1) can be determined. (Moments are per foot of wall.)

$$M_1 = \frac{13}{54} H_1^2 p = \frac{13}{54} * 7.0^2 * 1288 = 15194 \text{ lb} - \text{ft/ft}$$

and

$$
\begin{aligned}
MM_{(1,2,3)} &= \frac{1}{10}\left(\text{larger of } H_{(2,3,4)}\right)^2 p \\
&= \frac{1}{10}(10.75)^2 (1288) \\
&= 14884 \text{ lb} - \text{ft} / \text{ft}
\end{aligned}
$$

Hence,

Maximum moment M = 15194 lb - ft / ft (spacing OK for balanced mements)

3.2.4 Subgrade reaction using tributary method

Also, using the information contained in Figure 5.4b (Strom and Ebeling 2001), the subgrade reaction (R) is determined. (The subgrade reaction is expressed in pounds per foot of wall.)

$$
R = \left(\frac{3}{16} H_5\right) p \quad \text{(see Figure 5.4, Strom and Ebeling 2001)}
$$

i.e.,

$$
\begin{aligned}
R &= \frac{3}{16} * 10.75 * 1288 \\
&= 2596 \text{ lb / ft}
\end{aligned}
$$

3.2.5 Ground anchor load horizontal components

Again using the information contained in Figure 5.4b (Strom and Ebeling 2001), the horizontal component of each anchor load is determined. (Horizontal anchor loads T_i are also expressed in pounds per foot run of wall and design anchor force in pounds.) Assumed soldier spacing (s) = 6 ft.

Top tier:

$$
\begin{aligned}
T_1 &= \left(\frac{2}{3} H_1 + \frac{1}{2} H_2\right) p \\
&= \left(\frac{2}{3} * 7.0 + \frac{1}{2} * 10.75\right) 1288 \\
&= 12934 \text{ lb/ft}
\end{aligned}
$$

(Design anchor force = 12.934 kips/ft × 6 ft/cos 20° = 82.6 kips)

Second tier:

$$T_2 = \frac{1}{2}\left(H_2 + H_3\right)p$$
$$= \frac{1}{2}\left(10.75 + 10.75\right)1288$$
$$= 13846 \text{ lb/ft}$$

(Design anchor force = 13.846 kips/ft × 6 ft/cos 20° = 88.4 kips)

Third tier:

$$T_3 = \frac{1}{2}\left(H_3 + H_4\right)p$$
$$= \frac{1}{2}\left(10.75 + 10.75\right)1288$$
$$= 13846 \text{ lb/ft}$$

(Design anchor force = 13.846 kips/ft × 6 ft/cos 20° = 88.4 kips)

Lower tier:

$$T_4 = \left(\frac{H_4}{2} + \frac{23}{48}H_5\right)p$$
$$= \left(\frac{10.75}{2} + \frac{23 * 10.75}{48}\right)1288$$
$$= 13558 \text{ lb/ft}$$

(Design anchor force = 13.558 kips/ft × 6 ft/ cos 15° = 84.2 kips)

Use T_{max} = 13846 lb/ft (Spacing OK for approximately balanced T)

3.2.6 Soldier beam size

Assume soldier beam spacing (s) = 6.0 ft.

Note that the minimum permissible spacing is 4.0 ft (see Figure 8.5 of Strom and Ebeling 2001).

Hence, the maximum soldier beam design moment (M_{Max}) is

$$M_{Max} = \frac{15{,}194 * 6}{1000} = 91.2 \text{ ft - kip}$$

In accordance with Corps criteria (HQUSACE 1991), the allowable stresses for the soldier beams and wales shall be as follows:

Bending (i.e., combined bending and axial load): $f_b = 0.5 f_y$

Shear: $f_v = 0.33 f_y$

Allowable stresses are based on 5/6 of the AISC-ASD recommended values (AISC 1989) and reflect the Corps' design requirements for steel structures. Thus,

The allowable bending stress for Grade 50 steel: $F_b = 0.5\ F_y = 25$ ksi

Allowable shear stress for Grade 50 steel: $F_v = 0.33\ F_y = 16.5$ ksi

Required section modulus (S) for Grade 50 steel

Try 2 MC 10×25 Grade 50 steel for economical section.

$$S = \frac{M_{Max}}{F_b} = \frac{91.2 * 12}{25} = 43.8 \text{ in.}^3$$

From AISC (1989), HP 10×57 provides $S_{xx} = 58.8 > 43.8$ in.^{3} OK

or

2 MC 10×25 provides $S_{xx} = 44.0 > 43.8$ in.^{3} OK

Check shear capacity:

Maximum shear force, $V_{Max} = T_{max} * 6$ = 13846*6 = 83,076 lb = 83.1 kips

Required area, A = 83.1/16.5 = 5.04 in.^{2}

Shear area provided by 2 MC 10×25

$= 2*d*t_w$ =2*10*0.380 = 7.6 in.^{2} > 5.04 in.^{2} OK

where d and t_w are web depth and width for MC 10×25.

Use 2 MC 10×25 Grade 50 section.

3.2.7 Anchor lengths

For constructibility, the upper three tiers of ground anchors will be inclined downward at an angle of 20 deg, and the lower tier inclined downward at an angle of 15 deg (see Figure 3.2). Using the unbonded length requirements of Figure 8.5 (Strom and Ebeling 2001), the minimum unbonded length for each anchor tier can be determined. These calculations are provided in the following subsection (Section 3.2.7.1).

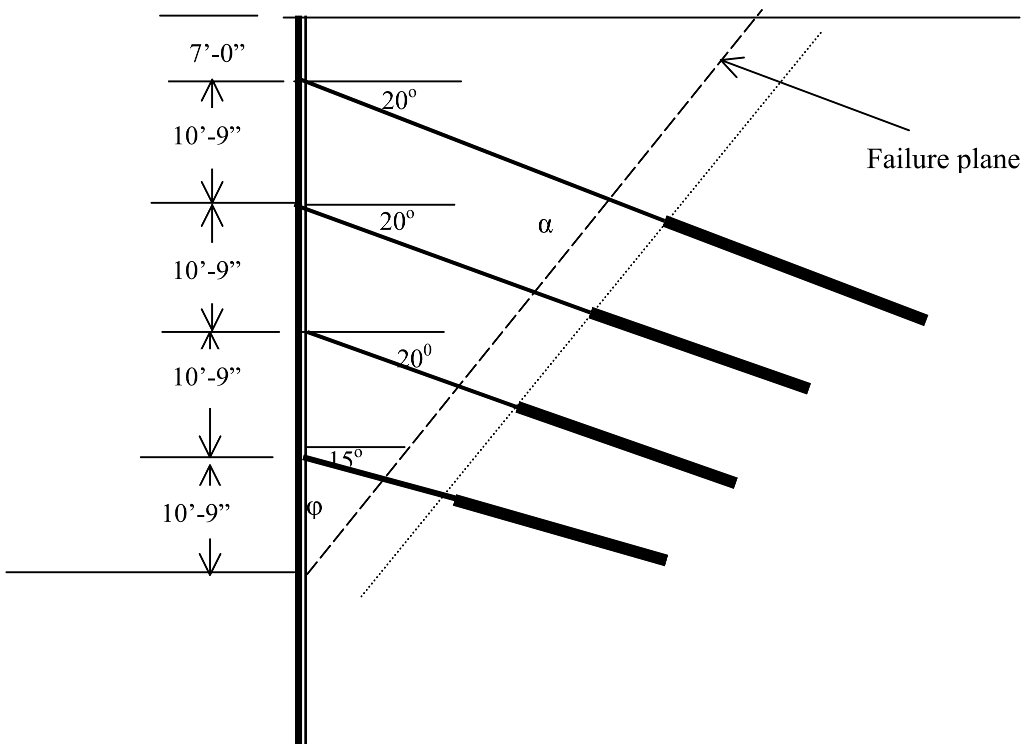


Figure 3.2 Anchors and placement

3.2.7.1 Unbonded anchor length, L_i. Assume 20-deg inclination for top three anchors and 15-deg inclination for bottom-tier anchor. The unbonded length must be sufficient such that anchor bond zone is beyond the short-term (undrained) and long-term (drained) potential failure surfaces and satisfy the Figure 8.5 Strom and Ebeling (2001) length criteria. With the short-term shear strength characterized in terms of S_u equal to 2,400 psf (with $\phi = 0$ deg), and with the long-term shear strength characterized in terms of ϕ' equal to 36 deg, the short-term loading condition will require greater unbonded anchor lengths. Thus, the potential failure plane will be based on the undrained shear strength with $\phi = 0$ deg (as is also done in the FHWA-RD-97-130 cohesive design example; refer to Figure 107).

$$\frac{\text{unbonded length, L}}{\varphi} = \frac{\text{height of anchor point}}{\alpha}$$

$$\varphi = 45° - \frac{\phi}{2} = 45° - 0° = 45°$$

$$\alpha = 180° - 70° - 45° = 65°$$

Top-tier anchor:

$$\frac{L}{\sin 45} = \frac{43.00}{\sin 65}$$

$$L = \frac{43.00 * \sin 45}{\sin 65} = 33.6 \text{ ft}$$

Allowing 5 ft or 0.2H clearance beyond shear plane (see Figure 8.5, Strom and Ebeling 2001),

$$L_1 = 33.6 + (0.2H \text{ or 5 ft whichever is greater})$$

$$= 33.6 + 10 = 43.6 \text{ ft} > 15 \text{ ft minimum required for strand anchor} \quad \text{OK}$$

(Minimum required for bar anchor is 10 ft)

Similarly, for the second-tier anchor:

$$L_2 = \frac{32.25}{43} * L + 0.2H$$

$$= 35.2 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

Third-tier anchor:

$$L_3 = \frac{21.5}{43} * L + 0.2\text{H}$$

$$= 26.8 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

Lower-tier anchor:

$$L_4 = \frac{10.75}{\sin 60} * \sin 45 + 0.2H$$

$$= 18.8 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

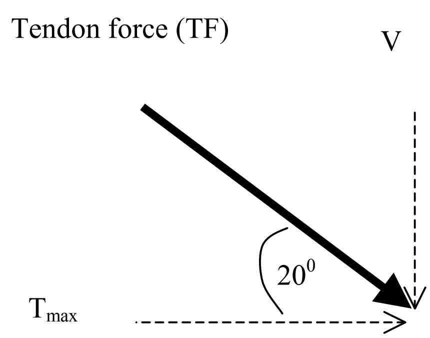
The unbonded length determined above should be verified using the internal stability analysis procedures for both undrained (i.e., short-term) and drained (i.e., long-term) conditions described in Strom and Ebeling (2002b). The verification process uses limiting equilibrium procedures, which can be performed by simple hand calculations or GPSS procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to meet internal stability performance requirements (i.e., factor of safety of 1.3 for a "safety with economy" design).

3.2.7.2 Bonded length of anchors, L_b. The usual practice is for the wall designer to specify the anchor capacity and any right-of-way and easement constraints required of the anchorage system. It is up to the tieback anchor contractor, usually a specialty sub-contractor, to propose the type of anchorage system to be used to meet the wall design requirements. A preliminary estimate of the bond length (L_b) required to develop the ground anchors is provided below.

The horizontal anchor forces T_2 and T_3 are all of equal magnitude and correspond to maximum horizontal anchor force T_{max} (Section 3.2.5). Because the horizontal anchor force T_1 and T_4 are within 7 percent and 2 percent, respectively, of this T_{max} value, the bond length computations will be made using the tendon force value of T_{max}. The computed bond length will be slightly conservative for anchor tendon 1.

With 6-ft spacing between soldier beams and using T_{max} = 13,846 lb/ft for all anchors, the maximum anchor (tendon) force TF is

$$TF = \frac{T_{max} * 6}{\cos 20°} = \frac{13846 * 6}{\cos 20°} = 88408 \text{ lb} \approx 88.4 \text{ kips}$$



An empirical method (Equation 1.2) was attempted to estimate the bond length of large-diameter straight-shaft gravity-grouted anchors for preliminary design purposes. Recall that it is up to the tieback anchor contractor, usually a specialty subcontractor, to propose the type of anchorage system to be used to meet the wall design requirements.

The design tendon force for the anchor bond zone is computed equal to 88.4 kips. Applying a factor of safety equal to 2.0 to this design force results in an ultimate anchor force equal to 176.8 kips.

No site-specific anchor load test data are available for use in this design. Consequently, preliminary design computations are made using traditional (non-site specific) assumptions for the range in value of the adhesion factor when computing an average ultimate soil-to-grout bond stress (Equation 1.3) and subsequently, the ultimate capacity of a large-diameter straight-shaft gravity-grouted anchor of 40-ft length (the maximum possible length without requiring specialized methods). The ultimate anchor capacity of a large-diameter straight-shaft gravity-grouted anchor was computed using Equation 1.2. These computations (not shown) indicate that a more robust anchorage system will be required to achieve an ultimate anchorage capacity of 176.8 kips.

Review of information contained in Section 1.5.4 indicates that a post-grouted (regroutable) ground anchor is a possible solution. (The anchor capacities cited for the case histories given and for the soil conditions cited in this section make this type of anchorage system a viable candidate.) For a ultimate anchor force equal to 176.8 kips, assuming a 40-ft bond length, the ultimate capacity of rate of load transfer corresponds to 4.42 kips per lineal ft. A preliminary bonded length L_b of 40 ft will be assumed for anchorage layout purposes. Recognize that the final bond length, anchor capacity, etc., will be confirmed by proof-testing and performance testing onsite (Strom and Ebeling 2002b).

3.2.7.3 Total anchor lengths ($Lt_i = L_i + L_b$).

Top-tier anchor:

$$Lt_1 = 43.6 + 40 = 83.6 \text{ ft} \approx 84 \text{ ft}$$

Second-tier anchor:

$$Lt_2 = 35.2 + 40 = 75.2 \text{ ft} \approx 76 \text{ ft}$$

Third-tier anchor:

$$Lt_3 = 26.8 + 40 = 66.8 \text{ ft} \approx 67 \text{ ft}$$

Lower-tier anchor:

$$Lt_4 = 18.8 + 40 = 58.8 \text{ ft} \approx 59 \text{ ft}$$

The total anchor length (bonded + unbonded length) determined above should be verified using the external stability analysis procedures described in Strom and Ebeling (2002b) for both short-term (i.e., undrained) and long-term (i.e., drained) conditions. This verification process also uses limiting equilibrium procedures, which can be performed by simple hand calculations or GPSS procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to prevent ground mass stability failure (i.e., meet external stability performance requirements with factor of safety of 1.3 for a safety with economy design).

3.2.8 Anchor strands

The number of 0.6-in.-diam ASTM A416, Grade 270, strands (ASTM 1999) required to meet "safety with economy" design requirements is determined. It is assumed that the final design force after losses will be based on an allowable anchor stress of 0.6 f_y, or 35.2 kips per strand.

Use the same maximum anchor load TF = 88.4 kips for sizing all four anchor strands (since $T_2 = T_3 = T_{max}$, and T_1 and T_4 are within 7 percent and 2 percent, respectively, of T_{max}).

From Table 8.5 (Strom and Ebeling 2002b),

Capacity of three 0.6-in. strands = 105.6 kips > 88.4 kips OK

Use three 0.6-in. strands.

3.2.9 Drilled-in shaft diameter

The drilled-in soldier beam will be fabricated from a pair of MC 10×25 shapes (using Grade 50 steel), as discussed in Section 3.2.6. Additionally, a 12-in.-diameter hole will be used to construct the anchor bond zone for all anchors, as discussed in Section 3.2.7.2.

The depth, d, and flange width, b_f, of an MC 10×25 are

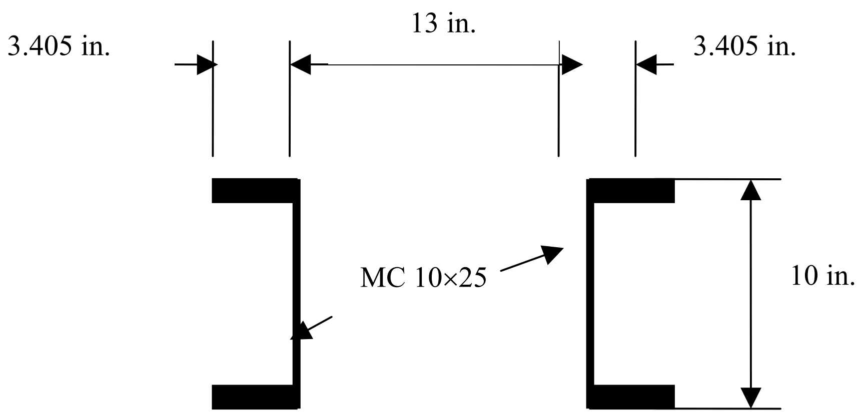
$$d = 10 \text{ in.}$$

and

$$b_f = 3.405 \text{ in.}$$

From Table 8.6, Strom and Ebeling (2001), trumpet diameter for three 0.6-in. strands and Case I corrosion protection = 5-7/8 in.

The distance between channels required is 13 in. to allow ample room for the installation of the anchor zone. For anchor zone details see Figure 10.2(b), Strom and Ebeling (2001).



The diameter for the drilled shaft (b) required to install the soldier beams is determined next.

For the previously described configuration of the pair of MC 10×28.5 shapes, the diagonal (from flange tip to flange tip) is given by

$$diagonal = \sqrt{d^2 + (2b_f + \text{clear spacing})^2}$$

$$diagonal = \sqrt{(10)^2 + (2 \bullet 3.405 + 13)^2}$$

$$diagonal = \sqrt{(10)^2 + (19.81)^2}$$

$$diagonal = 22.19 \text{ in.}$$

To install the fabricated pair of MC 10×25 shapes, the diameter of the drilled shaft (b) must be greater than the flange tip to flange tip diagonal of 22.19 in. Use 26-in.-diameter drilled shaft.

3.2.10 Temporary timber lagging

A temporary lagging design based on a uniform soil pressure distribution will be overly conservative since significant soil arching occurs behind soldier beam walls. Therefore, the size of the timber lagging is based primarily on experience or semi-empirical rules (see Table 12 of FHWA-SA-99-015 and Table 8.7 of Strom and Ebeling 2001).

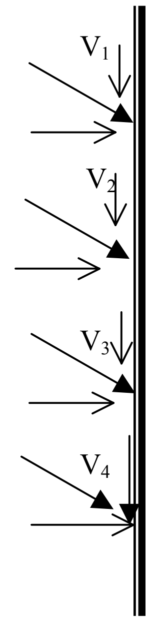
Clear lagging span ≈ soldier beam spacing = 6 ft

From Table 8.7 (Strom and Ebeling 2001),

for stiff clay the recommended thickness = 3 in.

Use 3-in. timber lagging

3.2.11 Soldier beam toe embedment



Soldier beam toe embedment requirements for both vertical and horizontal loads must be determined. With respect to the vertical component of prestress anchor load:

$$
\begin{aligned}
\sum V &= \left(V_1 + V_2 + V_3 + V_4\right) \\
&= \left[\left(T_1 + T_2 + T_3\right) * \tan 20° + T_4 * \tan 15°\right] * 6 \\
&= \left[\left(12934 + 13486 + 13486\right) * \tan 20° + 13558 * \tan 15°\right] * 6 \\
&= 108944 \text{ lb} = 108.9 \text{ kips}
\end{aligned}
$$

FHWA-SA-99-015 (page 95) general design recommendations for concrete backfill of predrilled holes include the use of structural concrete from the bottom of the hole to the excavation base and a lean-mix concrete for the remainder of the hole. The design concept is to provide maximum strength and load transfer in the permanently embedded portion of the soldier beam while providing a weak concrete fill in the upper portion, which can be easily removed and shaped to allow lagging installation. However, contractors often propose the use of lean-mix concrete backfill for the full depth of the hole to avoid the delays associated with providing two types of concrete in relatively small quantities. This design example follows the cohesive soil (stiff clay) design examples given in Section 10.2 of FHWA-RD-97-130 and in Example 2 of Appendix A of FHWA-SA-99-015, assuming that lean-mix concrete is used for the full depth of the hole.

The following computations are made to determine total force that the drilled-in shaft foundation must resist. A 6-ft depth of penetration is assumed in these computations for a 50-ft exposed wall height.

The total drilled-in soldier beam shaft weight assuming a 6-ft toe length is equal to the vertical component of anchor force plus the weight of a pair of MC 10×25 plus the weight of lean-mix concrete backfill plus the weight of timber lagging. The axial load transfer from the drilled shaft above the bottom of the wall to the retained soil, which acts upward on the soldier pile, is also included in the computations. The magnitude of each of these forces is summarized in the following six steps:

a. Vertical component of anchor force = 108.9 kips.

b. Compute the axial load transfer from the drilled shaft above the bottom of the wall to the retained soil:

(1) Axial load and ground movements are interrelated. The magnitude of the axial load depends upon the vertical components of the ground anchor loads, the strength of the supported ground, vertical and lateral movements of the wall, the relative movements of the ground with respect to the wall, and the axial load carrying capacity of the toe. FHWA-RD-98-066 (page 66) discusses results taken from walls in dense sands and stiff to hard clays in which the axial load measured in the soldier beam toes was <u>less</u> than the vertical

components of the ground anchors. This favorable axial load transfer from the soldier beam to the retained soil is idealized in Figure 41(b) of FHWA-RD-97-130. Axial load transferred to the ground above the bottom of the excavations in stiff to hard clays was equal to A_s times ($0.25S_u$) according to FHWA-RD-97-130 (page 88). A_s was the surface area of the soldier beam in contact with the ground above the bottom of the excavation, and $0.25S_u$ was the back-calculated adhesion. At the stiff cohesive sites, the load transferred from the soldier beam to the ground above the bottom of the excavation appears to be valid for the long-term condition. Using an adhesion equal to 25 percent of the undrained shear strength gives a lower load transfer rate than a rate based on drained shear strengths.

(2) To take advantage of this axial load transfer from the soldier beam to the retained stiff to hard (cohesive) soil, Table 11 in FHWA-RD-98-066 and Table 11 in FHWA-RD-97-130 stipulate that

$$S_u > \frac{\gamma \bullet H}{4} - 5.714 \bullet H$$

which for this problem becomes

$$2{,}400 \text{ psf} > \frac{132 \text{ pcf} \bullet 50 \text{ ft}}{4} - 5.714 \bullet 50 \text{ ft}$$

$$2{,}400 > 1{,}650 - 285.7$$

$$2{,}400 > 1{,}364.3 \qquad \mathit{OK}$$

(3) So the following set of computations assume that the axial load is transferred to the ground above the bottom of the excavation in this stiff clay site. Page 209 in FHWA-RD-97-130 gives this transfer force as

$$\text{Axial load transfer} = \alpha \bullet S_u \bullet A_s \bullet \left(H - H_1\right)$$

where H is the height of the wall (= 50 ft) and H_1 is the depth to the first row of anchors (=7 ft in Section 3.2.2). A_s is approximated as equal to half the circumference of the drilled-in soldier beam shaft,

$$A_s = \frac{1}{2} \bullet \pi \bullet d_s$$

With d_s equal to 26 in. (Section 3.2.9), A_s equals 40.84 in. (3.403 ft).

$$\text{Axial load transfer} = 0.25 \bullet 2400 \text{ psf} \bullet 3.403 \text{ ft} \bullet \left(50 \text{ ft} - 7 \text{ ft}\right)$$

$$= 600 \text{ psf} \bullet 3.403 \text{ ft} \bullet 43 \text{ ft} = 87{,}797 \text{ lb} = 87.8 \text{ kips}$$

(4) Note that this 87.8 kips force acts to <u>reduce</u> the axial load acting on the soldier beam foundation. The magnitude of this upward-acting force (from the perspective of the soldier beam) is significant. Great care must be exercised by the designers when taking advantage of this load transfer mechanism. It is assumed in this wall design that the soldier beam wall settles relative to the ground. Before applying this force in a design, designers should review the discussion and guidance given on pages 87-90 of FHWA-RD-97-130 and pages 66-69 in FHWA-RD-98-066. The instrumented wall case histories are discussed in FHWA-RD-98-066.

c. Weight of 2 MC 10×25 channels for 56-ft length = 2*0.025*56 = 2.8 kips

d. Compute the weight of lean-mix concrete backfill for a drilled-in soldier beam of length 56 ft:

(1) Weight of lean-mix concrete backfill for a drilled shaft diameter (d_s) of 26 in. and a drilled-in soldier beam cylinder of length 56 ft.

$$\text{Total area} = \pi \bullet \frac{d_s^{\,2}}{4}$$

$$\text{Total area} = \pi \bullet \frac{(26)^2}{4}$$

$$\text{Total area} = 530.93\ \text{in.}^2 = 3.687\ \text{ft}^2$$

$$\text{Gross weight} = \left(0.145\frac{\text{kips}}{\text{ft}^3}\right) \bullet \text{Total area} \bullet 56\ \text{ft}$$

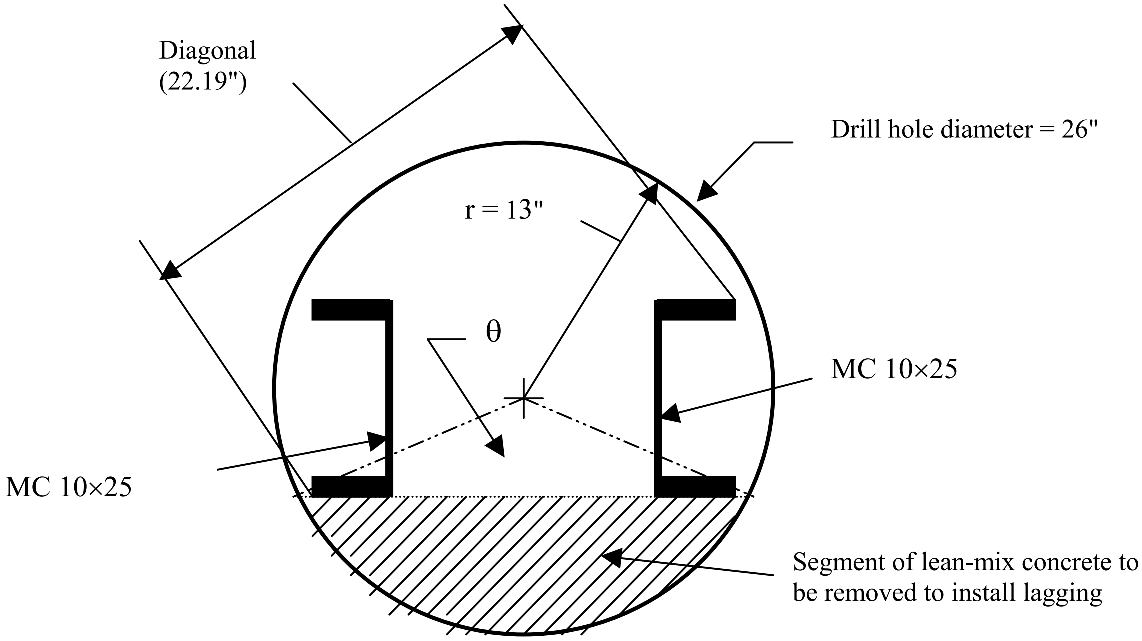
$$\text{Gross weight} = \left(0.145\frac{\text{kips}}{\text{ft}^3}\right) \bullet (3.687) \bullet 56$$

Gross weight = 29.93 kips

That is, the gross weight of a 56-ft-high, 26-in.-diameter lean-mix concrete cylinder is 29.93 kips. (This does not account for the weight of lean-mix concrete removed when placing the lagging.)

(2) Reduction in gross weight of a 56-ft-long cylinder for removal of the lean-mix concrete backfill during lagging installation.

- Compute the area of lean-mix concrete to be removed down the 50 ft (exposed) of height in front of the flanges of the pair of MC 10×25 shapes:



- Compute the segment (of circle) area in front of flanges to be removed:

$$\theta = 2 \bullet \left[\cos^{-1}\left(\frac{\text{half channel depth}}{\text{radius}} \right) \right] = 2 \bullet \left[\cos^{-1}\left(\frac{5}{13} \right) \right]$$
$$= 134.76 \text{ deg}$$

where

$$\text{half channel depth} = \frac{d}{2} = \frac{10}{2} = 5 \text{ in.}$$

$$r = \text{radius} = \frac{\text{diameter}}{2} = \frac{d_s}{2} = \frac{26}{2} = 13 \text{ in.}$$

$$\text{Segment area} = \pi r^2 \frac{\theta}{360} - r^2 \frac{\sin\theta}{2} = 138.75 \text{ in.}^2 \ = 0.96 \text{ ft}^2$$

The exposed wall height equals 50 ft. The area of lean-mix concrete to be removed down the 50 ft of exposed wall in front of the flanges for the pair of MC 10×25 shapes is equal to 0.963 ft^2 per ft of exposed wall height. For this 26-in.-diameter drilled-in shaft, the area removed represents approximately 26 percent of the cross-sectional area of the original 26-in.-diameter (cylinder) of lean-mix concrete per ft of exposed height.

- Compute the weight of lean-mix concrete removed during placement of lagging over the 50 ft of exposed height of wall:

$$\text{Weight removed} = \left(0.145\,\frac{\text{kips}}{\text{ft}^3}\right) \bullet \text{Segment area} \bullet H$$

$$\text{Weight removed} = \left(0.145\,\frac{\text{kips}}{\text{ft}^3}\right) \bullet 0.963\ \text{ft}^2 \bullet 50\ \text{ft} = 6.98\ \text{kips}$$

(3) Compute the weight of lean-mix concrete backfill for a drilled-in soldier beam of length 56 ft less the weight removed during placement of lagging:

Lean-mix net weight = Gross weight - Weight removed

Lean-mix net weight = 29.93 - 6.98 = 22.95 kips

e. Compute the weight of timber lagging over 50 ft exposed height for a span of 6 ft:

$$\text{Lagging weight} = \left(0.05\,\frac{\text{kips}}{\text{ft}^3}\right) \bullet \text{span} \bullet \text{height} \bullet \text{thickness}$$

$$\text{Lagging weight} = \left(0.05\,\frac{\text{kips}}{\text{ft}^3}\right) \bullet 6\ \text{ft} \bullet 50\ \text{ft} \bullet \left(\frac{3}{12}\right) = 3.75\ \text{kips}$$

f. Compute the applied total drilled-in soldier beam axial load:

$$Q_{applied} = \sum V - \text{Axial load transfer} + \text{Weight of channels} + \text{Lean mix net weight} + \text{Lagging weight}$$

$$Q_{applied} = 108.9\ \text{kips} - 87.8\ \text{kips} + 2.8\ \text{kips} + 22.95\ \text{kips} + 3.75\ \text{kips} = 50.6\ \text{kips}$$

Thus, for the 26-in.-diameter drilled-in shaft with a 6-ft depth of penetration, the applied axial load is equal to 50.6 kips.

3.2.12 Depth of toe penetration, D

This section outlines the depth of penetration computations. For a drilled-in shaft, Ultimate axial resistance = skin friction resistance + tip resistance

Hence:

$$Q_{ult} = Q_{skin} + Q_{tip} \quad \text{(see Equation 8.25 in Strom and Ebeling 2001)}$$

The factors of safety for axial capacity of drilled-in soldier beams in cohesive soil are

$FS_{skin} = 2.5$ and $FS_{skin} = 2.5$

according to Table 8.9 in Ebeling and Strom (2001) and Table 8.9 in FHWA-SA-99-015. Thus, the allowable axial load Q_{all} is given by

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} \left(\begin{array}{l} \text{modified form of Equation 8.18} \\ \text{in Strom and Ebeling 2001} \end{array} \right)$$

The traditional potential foundation failure mode due to axial loading is based on the assumption of the drilled-in shaft being fully effective in transferring the applied vertical load from the pair of steel channels through the lean-mix concrete mix to the surrounding soil. The corresponding traditional computation assumes the axial capacity is due to the drilled-in shaft acting as a single structural unit within the surrounding granular soil media. FHWA-SA-99-015 (page 95) and FHWA-RD-97-130 (page 90) note that for lean-mix backfilled drilled-in shafts, the lean-mix concrete may not be sufficiently strong to allow vertical load transfer from the soldier beam to the concrete. Consequently, a second potential failure mode must also be considered: The alternative potential failure mode assumes the soldier beam "punches" through the lean-mix, in which case the drilled-in shaft cross-section (assumed to be rectangular) will not be effective in transferring the load to the surrounding soil. Both potential failure modes are evaluated and the smallest capacity is used in the design. These computations are demonstrated in the following two sets of analyses. Note that in each set of computations a different value for depth of penetration D was used. Since the mechanisms for the two potential modes of failures are different, the minimum depth of penetration values required to satisfy the factors of safety against failure will also differ.

3.2.12.1 Analysis 1: Drilled-in shaft capacity (with shaft acting as single unit). In Section 3.2.9 the diameter of the drilled-in shaft was established to be 26 in. By trial and error using the following design analysis procedure it is determined that a depth of penetration (D) equal to 6 ft is required to meet the aforementioned factor of safety requirements. The soldier beam length is equal to 56 ft (= H + D = 50 ft + 6 ft).

a. *Ultimate skin friction.* The ultimate resistance due to skin friction, Q_{skin}, is given by

$$Q_{skin} = f_{skin} \bullet A_{cylinder}$$

The average unit skin friction is computed using the Strom and Ebeling (2001) Equation 8.28 to be

$$f_{skin} = \alpha \bullet S_u \quad \text{with the limitation that } f_{skin} < 5.5 \text{ ksf}$$

where α is equal to 0.55.

Thus,

$$f_{skin} = 0.55 \bullet 2.4 \text{ ksf} = 1.32 \text{ ksf}$$

The surface area of the drilled-in shaft is given by

$$A_{cylinder} = \pi \bullet (\text{diameter}) \bullet D$$

$$A_{cylinder} = \pi \bullet \left[26 \text{ in.} \bullet \left(\frac{\text{ft}}{12 \text{ in.}} \right) \right] \bullet 6 \text{ ft} = 40.841 \text{ ft}^2$$

Thus, the ultimate resistance due to skin friction along the 6-ft-long depth of penetration of the drilled-in shaft is

$$Q_{skin} = f_{skin} \bullet A_{cylinder} = 1.32 \bullet 40.841 = 53.9 \text{ kips}$$

b. *Ultimate tip resistance.* The ultimate tip resistance due to end bearing, Q_{tip}, is given by

$$Q_{tip} = q_b \bullet A_{tip}$$

The unit end bearing ultimate resistance is computed using the Strom and Ebeling (2001) Equation 8.29 as

$$q_b = N_c \bullet S_u \quad \text{with the limitation that } q_b < 80 \text{ ksf}$$

and

$$N_c = 6 \bullet \left[1 + 0.2 \bullet \left(\frac{\text{D}}{\text{diameter}} \right) \right] \quad \text{with the limitation that } N_c < 9$$

For the assumed depth D = 6 ft,

$$N_c = 6 \bullet \left[1 + 0.2 \bullet \left(\frac{6}{26/12} \right) \right] = 9.323$$

Use N_c equal to 9.

$$q_b = 9 \bullet 2.4 \text{ ksf} = 21.6 \text{ ksf}$$

The cross-sectional area of the tip is

$$A_{tip} = \pi \bullet \frac{(\text{diameter})^2}{4}$$

$$A_{tip} = \pi \bullet \frac{\left[26\text{ in.} \bullet \left(\frac{\text{ft}}{12\text{ in.}}\right)\right]^2}{4} = 3.687\text{ ft}^2$$

Thus, the ultimate tip resistance of the 26-in.-diameter drilled-in shaft is

$$Q_{tip} = q_b \bullet A_{tip} = 21.6 \bullet 3.687 = 79.6\text{ ksf}$$

c. *Ultimate axial load resistance.* The ultimate axial load Q_{ult} is computed to be

$$Q_{ult} = Q_{skin} + Q_{tip} = 53.9 + 79.6 = 133.5\text{ kips}$$

Note that skin friction provides 40 percent of the ultimate axial load resistance, while tip resistance due to end bearing provides 60 percent of this ultimate axial load value.

d. *Allowable axial load.* The allowable axial load Q_{all} is computed to be

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} = \frac{53.9}{2.5} + \frac{79.6}{2.5} = 21.6 + 31.8 = 53.4\text{ kips}$$

Note that skin friction provides 40 percent of the allowable axial load resistance, while tip resistance due to end bearing provides 60 percent of this allowable axial load value.

The allowable axial load Q_{all} for this 26-in.-diameter drilled-in shaft with an assumed 6-ft depth of embedment is 53.4 kips, which is 2.8 kips larger than the applied axial load of 50.6 kips (see Section 3.2.11), i.e., $Q_{applied} > Q_{all}$. Thus, a 6-ft depth of penetration is acceptable for this assumed potential mode of foundation failure. Recall that this potential mode of foundation failure assumes the shaft acting as single unit.

Solution process: The procedure used to determine the depth of penetration D in this problem was as follows: assume a depth of penetration; compute the ultimate axial load resistance Q_{ult}; compute the allowable axial load Q_{all}; compute the total applied axial load for the drilled-in soldier beam system $Q_{applied}$ (Section 3.2.11); adjust the depth of penetration D as necessary and repeat computations until Q_{all} is approximately equal to $Q_{applied}$. Ensure that for the final value of D used in the computations Q_{all} is greater than or equal to $Q_{applied}$.

3.2.12.2 Analysis 2: "Punching" soldier beams capacity. The drilled-in soldier beam backfilled with a lean-mix concrete has the potential not to act as a single structural unit in which the axial load is transferred from the pair of channels through the lean-mix to the surrounding soil foundation but where the pair of channels "punches" through the lean-mix concrete backfill. In Section 3.2.9 the diameter of the drilled-in shaft was established to be 26 in. By trial and error using the following design analysis procedure it is determined that a depth of penetration (D) equal to 4 ft is required to meet established factor of safety requirements. (Note that this value of D differs from the 6-ft value used in Analysis 1 computations. The authors of this report are demonstrating that the minimum required depth of penetration is not the same for the two different types of failure modes.) For the Analysis 2 procedure the soldier beam length is equal to 54 ft (= H + D = 50 ft + 4 ft). As an alternative the designer could verify the depth of penetration established by the design Analysis 1 procedure satisfies factor of safety requirements for the Analysis 2 procedure.

The following computations assume the pair of soldier beam channels will punch through the lean-mix concrete backfill rather than transfer the load through the backfill to the ground. Note that in these computations the diameter of the drilled shaft is not used (i.e., the drilled shaft cross-section will not be effective in transferring the load to the surrounding soil). Instead, the rectangular "box" perimeter of the pair of channels is used in both the skin friction and tip resistance computations.

a. *Ultimate skin friction.* The ultimate resistance due to skin friction, Q_{skin}, is given by

$$Q_{skin} = f_{skin} \bullet A_{box}$$

The average unit skin friction for this "punching" mode of failure is computed using the Strom and Ebeling (2001) Equation 8.20 to be

$$f_{skin} = K \bullet \sigma'_{ave} \bullet \tan(\delta)$$

with

$$\sigma'_{ave} = \gamma \bullet \left(\frac{H + D}{2}\right) = 132 \bullet \left(\frac{50 + 4}{2}\right) = 3564 \text{ psf}$$

FHWA-RD-97-130 (page 94) and FHWA-SA-99-015 (page 95) note that when a lean-mix concrete backfill is used in a drilled-in shaft, f_{skin} is computed using K = 2 and δ = 35 degrees in the f_{skin} equation (see page 180 in Strom and Ebeling 2001). Note that these parameter values are specific to the "punching" mode of failure through the lean-mix concrete. Thus, f_{skin} becomes

$$f_{skin} = 2 \bullet \left[3564 \text{ psf} \bullet \left(\frac{\text{kips}}{1000 \text{ lb}}\right)\right] \bullet \tan(35) = 4.991 \text{ ksf}$$

The surface area of the rectangular "box" defined by the perimeter of the pair of channels is given by

$$A_{box} = [2 \bullet (\text{channel depth}) + 2 \bullet (\text{flange - to - flange width})] \bullet D$$

$$A_{box} = \left[2 \bullet (\text{channel depth}) + 2 \bullet \begin{pmatrix} 2 \bullet b_f + \text{clear space} \\ \text{between channels} \end{pmatrix} \right] \bullet D$$

$$A_{box} = [2 \bullet (10 \text{ in.}) + 2 \bullet (2 \bullet 3.405 \text{ in.} + 13 \text{ in.})] \bullet \left(\frac{1}{12} \right) \bullet 4 \text{ ft}$$

$$= [(20 \text{ in.}) + (39.62 \text{ in.})] \bullet \left(\frac{1}{12} \right) \bullet 4 \text{ ft} = 19.873 \text{ ft}^2$$

Thus, the ultimate resistance due to skin friction along the 4-ft-long depth of penetration of the drilled-in shaft is

$$Q_{skin} = f_{skin} \bullet A_{box} = 4.991 \text{ ksf} \bullet 19.873 = 99.19 \text{ kips}$$

b. *Ultimate tip resistance.* The ultimate tip resistance due to end bearing, Q_{tip}, is given by

$$Q_{tip} = q_b \bullet A_{tip}$$

The unit end bearing ultimate resistance is computed using the Strom and Ebeling (2001) Equation 8.29 relationship

$$q_b = N_c \bullet S_u$$

where for a rectangle

$$N_c = 5 \bullet \left\{ 1 + 0.2 \bullet \left[\frac{D}{(2 \bullet b_f + \text{clear spacing})/12} \right] \right\} \bullet \left[1 + 0.2 \bullet \left(\frac{D}{d} \right) \right]$$

with the limitation that N_c is less than 7.5 to 9. Recall from Section 3.2.9 that b_f is 3.405 in., clear spacing is 13 in. and d is 10 in.

$$N_c = 5 \bullet \left\{ 1 + 0.2 \bullet \left[\frac{4}{(2 \bullet 3.405 + 13)/12} \right] \right\} \bullet \left[1 + 0.2 \bullet \left(\frac{4}{10/12} \right) \right] = 14.55$$

Use N_c equal to 9.

This results in a unit end bearing ultimate resistance equal to

$$q_b = 9 \bullet 2.4 \text{ ksf} = 21.6 \text{ ksf}$$

The cross-sectional area of the rectangular "box" tip is

$$A_{tip} = (\text{channel depth}) \bullet (\text{flange - to - flange width})$$

$$A_{tip} = (\text{channel depth}) \bullet (2 \bullet b_f + \text{clear space between channels})$$

$$A_{tip} = [(10 \text{ in.}) \bullet (2 \bullet 3.405 \text{ in.} + 13 \text{ in.})] = [(10 \text{ in.}) \bullet (19.81 \text{ in.})]$$
$$= 198.1 \text{ in.}^2$$

Thus, the ultimate tip resistance of the 26-in.-diameter drilled-in shaft is

$$Q_{tip} = q_b \bullet A_{tip} = 21.6 \bullet 198.1 \bullet \left(\frac{1}{144}\right) = 29.7 \text{ ksf}$$

c. *Ultimate axial load resistance.* The ultimate axial load Q_{ult} is computed to be

$$Q_{ult} = Q_{skin} + Q_{tip} = 99.19 + 29.7 = 128.9 \text{ kips}$$

Note that skin friction provides 77 percent of the ultimate axial load resistance, while tip resistance due to end bearing provides 23 percent of this ultimate axial load value.

d. *Allowable axial load.* The allowable axial load Q_{all} is computed to be

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} = \frac{99.19}{2.5} + \frac{29.7}{2.5} = 39.68 + 11.88 = 51.56 \text{ kips}$$

Note that skin friction provides 77 percent of the allowable axial load resistance, while tip resistance due to end bearing provides 23 percent of this allowable axial load value.

The allowable axial load Q_{all} for this 26-in.-diameter drilled-in shaft with an assumed 4-ft depth of embedment is 51.56 kips, which is 2.09 kips larger than the applied axial load of 49.47 kips (computations not shown but follow those made in Section 3.2.11 using a 4-ft depth of penetration). Thus, a 4-ft depth of penetration is acceptable for this assumed potential mode of foundation failure. Recall that this potential mode of foundation failure assumes the soldier beam "punches" through the lean-mix concrete backfill.

Solution process: The procedure used to determine the depth of penetration D in this problem was as follows: assume a depth of penetration; compute the ultimate axial load resistance Q_{ult}; compute the allowable axial load Q_{all}; compute the total applied axial load for the drilled-in soldier beam system $Q_{applied}$ (following the procedure outlined in Section 3.2.11); adjust the depth of penetration D as necessary; and repeat computations until Q_{all} is approximately equal to $Q_{applied}$. Ensure that for the final value of D used in the computations Q_{all} is greater than or equal to $Q_{applied}$.

3.2.12.3 Concluding remarks. The minimum required depths of penetration were computed in this section for two potential failure modes. It was found in design Analysis 1 that a 6-ft minimum depth of penetration is required to be safe by the traditional potential foundation failure mode in which the drilled-in shaft acts as a single structural unit within the surrounding soil media. It was found in design Analysis 2 that a 4-ft minimum depth of penetration is required for the system to be safe against the alternative potential failure mode that assumes the soldier beam "punches" through the lean-mix. Therefore, the required depth of penetration D for this drilled-in shaft retaining structure is 6 ft for axial load considerations. Note that a high percentage of the axial capacity is being carried by end bearing in Analysis 1. Discussion contained in Chapter 6 of FHWA-RD-97-130 (or Chapter 8 in Strom and Ebeling 2001) should be reviewed prior to finalizing the depth of penetration D at 6 ft for axial loading in light of this observation.

3.2.13 Lateral capacity of soldier beam toe

Assume, based on vertical load requirements, the final toe penetration (D) is 6.0 ft.

Check lateral capacity of soldier beam toe:

Subgrade reaction R = 2,596 lb/ft (Section 3.2.4)

Total toe reaction = 2,596*6= 15,576 lb = 15.6 kips/soldier beam

A spreadsheet incorporating the Wang-Reese passive resistance equations (Table 3.1) is used to determine lateral resistance of the soldier beam toe following the procedure outlined in Section 8.7 of Strom and Ebeling (2001) or Section 6.2 in FHWA-RD-97-130. Note that the soldier beam width of 1.65 ft (19.81 in.) is used for drilled shafts backfilled with lean concrete per FHWA-RD-97-130 (page 111) recommendations. If structural concrete is used to backfill the shaft, then the drilled shaft diameter (26 in.) would be used in the computations. Alpha and beta in this table are angles used to define the three-dimensional geometrical configuration of the "passive" failure wedge developing in front of the soldier beam on the excavated side (refer to Figure 8.6 in Strom and Ebeling 2001). The Wang-Reese definition for β is

$$\beta = 45 + \frac{\phi'}{2}$$

Table 3.1
Spreadsheet for Computing Passive Resistance for Clay (“Safety with Economy” Design)

INPUT VARIABLES

unit weight	wall height	beam width	beam spacing	Su	Sc	depth disturbance	toe depth	toe reaction
0.132	50	1.65	6	2.4	4.3500	0	6	15.6

SPREADSHEET FOR EVALUATING TOE CAPACITY FOR CLAY (“safety with economy” design)

Toe depth,D (ft)	*(Eq 6.20) Scr (ft)*	(Eq 6.19) *single beam wedge resistance (kip/ft)*	(Eq 6.21) *row beams wedge resistance (kip/ft)*	*critical wedge resistance (kip/ft)*	(Eq 6.23) *single beam flow resistance (kip/ft)*	*critical wedge/flow resistance (kip/ft)*	(Eq 6.24) *Rankine passive resistance (kip/ft)*	*passive resistance (kip/ft)*	*Failure Mode*	*total passive force at given toe depth (kips)*	*allowance for disturbance (Eq 6.12) for depth*	*net passive force at given depth*	*factor of safety*
col.1	***col 2***	***col 3***	***col 4***	***col 5***	***col 6***	***col 7***	***col 8***	***col 9***	***col 10***	***col 11***	***col 12***	***col13***	***col 14***
0	0.0000	7.9200	39.2400	7.9200	43.5600	7.9200	28.8000	7.9200	Wedge	0.0000	0.0000	0.0000	0.0000
1	0.4671	14.9298	40.0320	14.9298	43.5600	14.9298	29.5920	14.9298	Wedge	11.4249	0.0000	11.4249	0.7324
2	0.9257	21.9396	40.8240	21.9396	43.5600	21.9396	30.3840	21.9396	Wedge	29.8596	0.0000	29.8596	1.9141
3	1.3762	28.9494	41.6160	28.9494	43.5600	28.9494	31.1760	28.9494	Wedge	55.3041	0.0000	55.3041	3.5451
4	1.8186	35.9592	42.4080	35.9592	43.5600	35.9592	31.9680	31.9680	Wedge	85.7628	0.0000	85.7628	5.4976
5	2.2534	42.9690	43.2000	42.9690	43.5600	42.9690	32.7600	32.7600	Wedge	118.1268	0.0000	118.1268	7.5722
6	**2.6806**	**49.9788**	**43.9920**	**43.9920**	**43.5600**	**43.5600**	**33.5520**	**33.5520**	**Wedge**	**151.2828**	**0.0000**	**151.2828**	**9.6976**
7	3.1004	56.9886	44.7840	44.7840	43.5600	43.5600	34.3440	34.3440	Wedge	185.2308	0.0000	185.2308	11.8738
8	3.5130	63.9984	45.5760	45.5760	43.5600	43.5600	35.1360	35.1360	Wedge	219.9708	0.0000	219.9708	14.1007
9	3.9187	71.0082	46.3680	46.3680	43.5600	43.5600	35.9280	35.9280	Wedge	255.5028	0.0000	255.5028	16.3784
10	4.3176	78.0180	47.1600	47.1600	43.5600	43.5600	36.7200	36.7200	Wedge	291.8268	0.0000	291.8268	18.7068
11	4.7098	85.0278	47.9520	47.9520	43.5600	43.5600	37.5120	37.5120	Wedge	328.9428	0.0000	328.9428	21.0861
12	5.0955	92.0376	48.7440	48.7440	43.5600	43.5600	38.3040	38.3040	Wedge	366.8508	0.0000	366.8508	23.5161
13	5.4749	99.0474	49.5360	49.5360	43.5600	43.5600	39.0960	39.0960	Wedge	405.5508	0.0000	405.5508	25.9968
14	5.8482	106.0572	50.3280	50.3280	43.5600	43.5600	39.8880	39.8880	Flow	445.0428	0.0000	445.0428	28.5284
15	6.2154	113.0670	51.1200	51.1200	43.5600	43.5600	40.6800	40.6800	Flow	485.3268	0.0000	485.3268	31.1107

With undrained conditions (i.e., short-term load case) within the cohesive soil, β is equal to 45 degrees and α is set equal to 0 degrees.

Sc in this table is the clear span between piles. Sc is 4.35 ft, equal to the span(s) of 6 ft minus the soldier beam width of 1.65 ft (19.81 in.).

Passive resistance back-calculated for soldier beam and lagging systems compares favorably with passive resistance equations developed by Wang and Reese (1986). Several passive failure mechanisms must be evaluated for each increment of soldier beam embedment and the pressure associated with the governing failure condition summed over each increment of depth to determine the soldier beam total passive resistance. The failure mechanism evaluation and summing process for the "safety with economy" design is summarized in Table 3.1 (and for the "stringent displacement control" design in Section 3.3.13). In Table 3.1, the pressures attributed to the various failure mechanisms are provided in columns 5 through 8, and the pressures associated with the governing failure condition are indicated in column 9. The equation numbers referenced in the various columns of the table refer to equations from FHWA-RD-97-130. Similar equations can be found in FHWA-SA-99-015 and in Strom and Ebeling (2001). Table 3.2 provides the reference equation numbers associated with each of these three references.

Table 3.2
Equation References for Passive Resistance Calculations Stated in Tables 3.1 and 3.4

Column No.	Description of Equation	Reference Document–Equation Number		
		FHWA-RD-97-130	FHWA-SA-99-015	Strom and Ebeling (2001)
3	Single beam wedge resistance	Eq. 6.19	Eq. B-8	Eq. 8.13
4	Intersecting wedge resistance	Eq. 6.21	Eq. B-10	Eq. 8.15
6	Single-beam flow resistance	Eq. 6.23	Eq. B-11	Eq. 8.16a
7	Rankine passive resistance	Eq. 6.24	Eq. B-13	Eq. 8.17

The computations summarized in Table 3.1 are for the 50-ft-high tieback wall in stiff clay. These computations explicitly follow those given in the Figure 113 spreadsheet procedure of FHWA-RD-97-130 (page 212). The soil properties (S_u = 2,400 psf, γ = 132 psf) used for the 50-ft-high wall are the same as those of FHWA-RD-97-130. The differences in the results (that is between Table 3.1 and Figure 113 of the FHWA report) are due to the soldier beam width (1.65 ft in Table 3.1 versus 1.778 ft in FHWA Figure 113). In accordance with the FHWA report, Table 3.1 does not include the total active force reduction used in the granular soil examples. The author of FHWA-RD-97-130 states (on page 109) that "the Wang and Reese equations for clays do not include an active pressure term. In stiff clays the active pressure may be negative behind the wall. Considering negative pressures during design is not reasonable since the soldier beam will move away from the soil." Further, the FHWA report states, "a continuous wall will normally be used when active pressures are positive."

Table 3.1 shows that the soldier beams, 6 ft on centers with a toe penetration (D) of 6.0 ft, have a lateral resistance of 151.28 kips, and the factor of safety equals 9.7. In this design problem, the depth of penetration is controlled by axial load considerations.

As stated previously, Table 3.1 (as per Figure 133 of FHWA-RD-97-130) does not include computations for "total active force reduction." These computations are performed below using Equation 6.25 of the FHWA report. For tall soldier beam walls, the computations will result in a somewhat lower net passive force and lower factor of safety.

$$P_{active} = \gamma_{ave}(H + D) - 2S_u \qquad \text{Equation 6.25 (FHWA-RD-97-130)}$$

At the elevation corresponding to bottom of the excavation where the depth of penetration (D) is equal to zero, the active earth pressure (behind the soldier beam and below the retained side soil) is

$$P_{D=0} = \left[132(50+0) - 2(2400)\right]\frac{1}{1000} = 1.80 \text{ ksf}$$

At the toe of the soldier beam where the depth of penetration (D) is equal to 6.0 ft, the active pressure is

$$P_{D=6} = \left[132(50+6) - 2(2400)\right]\frac{1}{1000} = 2.59 \text{ ksf}$$

The total active force reduction (P_{AFR}) for soldier beams with a width (b) of 1.65 ft and a depth of embedment (D) of 6.0 ft is

$$P_{AFR} = (1.65)\,(6.0)\,(1.80 + 2.59)\,(0.5) = 21.73 \text{ kips per soldier beam}$$

Therefore, for a depth of penetration equal to 6 ft (column 1, Table 3.1), the net passive resistance (column 13, Table 3.1) is 151.28 - 21.73 = 129.55 kips, which reduces the factor of safety from 9.7 (column 14, Table 3.1) to 8.3. Since this factor of safety is still greater than 1.5, it can be assumed the lateral capacity of the soldier beam toe is more than adequate for a "safety with economy" design.

The authors of this report recommend that designers always consider positive active earth pressures and the effect they have in reducing net toe resistance. Recall that the focus of this report is tall tieback walls. In general, the taller the wall, the more likely it is that positive active earth pressures may be encountered in stiff clays. For this particular clay site, assuming a penetration depth of 6 ft, positive active earth pressure will begin to occur when the wall height reaches [2 (2,400)/132] - 6 = 30.36 ft. The designer should also consider the cautionary advice provided in FHWA-RD-97-130 with respect to the use of soldier beam systems under "positive active earth pressure conditions."

A summary of results for the 50-ft-high soldier beam safety with economy design is given in Table 3.3.

Table 3.3 Summary of Results for Four-Tier, 50-ft Drilled-In Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Retaining Cohesive Soil–“Safety with Economy” Design		
Parameter		**Value**
Wall height		50 ft
Soldier beam spacing		6 ft
Soldier beam design moment		91.2 kip-ft
Soldier beam size		2 MC10×22
Soldier beam length		56 ft
Drill shaft diameter		26 in.
Toe reaction		15.6 kips
Top-tier anchor	H_1	7 ft, 0 in.
	Anchor inclination	20 deg
	Design load	82.6 kips
	Unbonded length	43.6 ft
	Bonded length	40 ft
	Total length	84 ft
	Tendon size	three 0.6-in. strands
Second-tier anchor	H_2	10 ft, 9 in,
	Anchor inclination	20 deg
	Design load	88.4 kips
	Unbonded length	35.2 ft
	Bonded length	40 ft
	Total length	76 ft
	Tendon size	three 0.6-in. strands
Third-tier anchor	H_3	10 ft, 9 in.
	Anchor inclination	20 deg
	Design load	88.4 kips
	Unbonded length	26.8 ft
	Bonded length	40 ft
	Total length	67 ft
	Tendon size	three 0.6-in. strands
Lower-tier anchor	H_4	10 ft, 9 in.
	Anchor inclination	15 deg
	Design load	84.2 kips
	Unbonded length	18.8 ft
	Bonded length	40 ft
	Total length	59 ft
	Tendon size	three 0.6-in. strands

3.2.14 Basal stability

The apparent pressure diagram used to compute the horizontal components of the anchor forces and the horizontal subgrade reaction force is for the condition where the soil at the bottom of the wall is not near a state of plastic equilibrium (i.e., failure), as discussed in Section 4.2.2 of FHWA-RD-97-130 and Section 5.8.2 of FHWA-SA-99-015. For a wide, infinitely long excavation in a homogeneous soft to medium clay of constant shear strength, the factor of safety is given by

$$FS = \frac{N_c}{\gamma \bullet \frac{H}{S_u}} = \frac{5.14}{N_s}$$

where

γ is the total unit weight

N_s is the stability number

Recall the stability number N_s has been used to identify excavation support systems with potential for movement and basal heave problems (Table 8.7 in Strom and Ebeling 2001; Table 12 in FHWA-SA-99-015). In this problem N_s is

$$N_s = \gamma \bullet \frac{H}{S_u} = 0.132 \bullet \frac{50}{2.4} = 2.75$$

Small values of N_s, with respect to a value of 5.14, indicate basal stability and small ground movements. The factor of safety against basal heave is

$$FS = \frac{5.14}{N_s} = \frac{5.14}{2.75} = 1.87$$

Current practice according to FHWA-RD-97-130 is to use a minimum factor of safety of 1.5 against basal heave. A more detailed discussion is presented in Sections 54.3 and 37.3.1 of Terzaghi, Peck and Mesri (1996). Cacoilo, Tamaro, and Edinger (1998) indicate a minimum safety factor of 1.5 is desirable to limit soil displacements. FHWA-SA-99-015 (page 107) notes that as the factor of safety decreases, loads on the lowest anchor increase. FHWA-SA-99-015 suggests a minimum factor of safety against basal heave of 2.5 for permanent facilities and 1.5 for support excavation facilities. Factors of safety below these target values indicate that more rigorous procedures such as limit equilibrium methods or Henkel's method should be used to evaluate design earth pressures loadings according to FHWA-SA-99-015 (page 107).

For weak soils (soft clays and loose sands) the failure surface will extend below the bottom of the cut, rather than through bottom corner of the cut (FHWA-RD-98-065). For clay soils the deeper failure plane condition (i.e., below the bottom of the cut condition) can be evaluated by the Bishop method. The Bishop method is in reasonable agreement with the Henkel method (FHWA-SA-99-015). The Bishop method can be used in a general-purpose slope stability program (GPSSP) to determine the total load the tieback system must carry to meet the factor of safety requirements established for the project. The total load determined from a Bishop method internal stability limiting equilibrium analysis can be redistributed into an apparent pressure diagram. This apparent pressure diagram should be used as a basis for design, if it provides a greater total load than that obtained from either a conventional apparent pressure diagram that assumes a "bottom corner of the cut" failure condition, or from an apparent pressure diagram constructed for the drained (long-term) condition. GPSSP analyses are described in FHWA-RD-98-065 and in Strom and Ebeling (2002b). GPSSP analyses should always be used to verify that the total load required to meet internal stability safety requirements is equal to or less than that used for the original design.

As the soil above the failure plane attempts to move out, shear resistance is mobilized in the soldier beams. Shear in the soldier beams provides additional resistance to soil movement. Predicting soldier beam shear requires the consideration of three possible failure modes: (1) shear in the soldier beam; (2) flow of the soil between the soldier beams: and (3) lateral capacity of the soldier beams. These three failure mechanisms are discussed in Section 4.3.3 of FHWA-RD-98-065. The resistance provided by the soldier beams for each of the three possible failure modes can be estimated and included in a GPSSP analysis, provided the GPSSP used is capable of modeling the soldier beams as reinforcement.

The calculated mass stability (i.e., external stability) slip circles for soft clays can be deep, and are generally located beyond or at the end of the tieback anchorage zone (Cacoilo, Tamaro, and Edinger 1998). Possibly, ground mass stability can be improved by increasing the depth of the soldier beam or by extending the length of the tiebacks, although significant improvement in the factor of safety can sometimes be difficult to obtain by these methods (Cacoilo, Tamaro, and Edinger 1998).

3.3 "Stringent Displacement Control" Design

For the Corps' stringent displacement control design, a limiting equilibrium approach is used, with a factor of safety of 1.5 applied to the shear strength of the soil. The total earth pressure load (P_{tl}) is then determined based on the limiting equilibrium analysis. Limiting equilibrium calculations for the stringent displacement control design are provided below. This process produces an EPF equal to 26.0 pcf, compared to an EPF of 22.7 pcf determined by the previous limiting equilibrium analysis for the "safety with economy" design (Section 3.2) using drained strength parameters (i.e., long-term condition). The total earth pressure load (P_{tl}) is determined assuming the shear strength of the soil is factored by the target factor of safety such that

$$\phi_{mob} = \tan^{-1}(\tan\phi / FS)$$

Accordingly,

$$\phi_{mob} = \tan^{-1}\left(\frac{\tan\phi}{1.5}\right)$$

$$= \tan^{-1}\left(\frac{\tan 36°}{1.5}\right) = 25.8°$$

$$K_A = \tan^2\left(45° - \frac{\phi_{mob}}{2}\right) = 0.394$$

$$P = K_A \gamma \frac{H^2}{2} = 0.394 * 132 * \frac{50^2}{2} = 65010 \text{ lb/ft}$$

$$\text{Effective pressure factor, EPF} = \frac{P}{H^2} = \frac{65010}{50^2} = 26 \text{ pcf}$$

An EPF value of 26 pcf is used in the construction of the apparent earth pressure diagram and in all subsequent computation of the prestress design anchor forces.

3.3.1 Anchor points

One of the intended purposes of installing a tieback wall is to restrict wall and retained soil movements during excavation to a tolerable movement so that adjacent structures will not experience any distress. If a settlement-sensitive structure is founded on the same soil used for supporting the anchors, a tolerable ground surface settlement may be less than 1/2 in. according to FHWA-RD-81-150. FHWA-RD-81-150 also states that if the adjacent structure has a deep foundation that derives its capacity from a deep bearing stratum not influenced by the excavation activity, settlements of 1 in. or more may be acceptable. Obviously, this guidance is geared toward situations involving buildings that are adjacent to the excavation. Figure 75 in FHWA-RD-97-130 gives settlement profiles/envelopes behind flexible walls in different soils.

Wall and retained soil movements predictions are based on experience. Several types of movements are associated with flexible anchored walls. These are described on page 120 of FHWA-SA-99-015. Movement can occur due to (1) wall cantilever action associated with installation of the first anchor; (2) wall bulging actions associated with subsequent excavation stages and anchor installations; (3) wall settlement associated with mobilization of end bearing; (4) elastic elongation of the anchor tendons associated with a load increase; (5) anchor yielding or load redistribution in the anchor bond zone; and (6) mass ground movements behind the tieback anchors. The last three components of deformation result in translation of the wall and are generally small for anchored walls constructed in competent soils according to FHWA-SA-99-015. Typical lateral and horizontal movements for flexible retaining walls have been presented by Peck (1969), FHWA-RD-75-128 (1975), and Clough and O’Rourke (1990). FHWA-RD-97-130 states that maximum lateral movements in ground suitable for permanent ground anchor walls are generally less than 0.005H, with average maximum movements of about 0.002H. For a 50-ft-high wall the average maximum horizontal movement would be 1.2 in. by this relationship. FHWA-RD-97-130 also states that maximum vertical settlements in ground suitable for permanent ground anchor walls are generally less than 0.005H, with average maximum settlement tending toward 0.0015H. Maximum settlement occurs near the wall. For a 50-ft-high wall the average maximum settlement would be 0.9 in. by this relationship. Note that actual wall performance and especially horizontal and vertical deformations are a function of both design and construction details.

Lateral wall movements and ground settlements cannot be eliminated for flexible tieback walls. However, they can be reduced by (1) controlling soldier beam bending deformations (i.e., cantilever and bulging displacements); (2) minimizing soldier beam settlements by installing the tieback anchors at flat angles (note that grouting of anchors installed at angles less than 10 degrees from horizontal is not common unless special grouting techniques are used) and properly designing the embedded portion of the wall to carry applied axial loads; and (3) increasing the magnitude of the anchor design forces for which the anchors are prestressed to over that obtained in a "safety with economy" design (given in Section 3.2).

Among the factors contributing to bending deformations are (1) the depth of excavation prior to installation and prestress of the first row of anchors, and (2) the span between the subsequent, lower rows of anchors. FHWA-RD-97-130 and others observe that reducing the distance to the upper ground anchor will reduce the cantilever bending deformations. The magnitude of this deformation, which occurs prior to installation of the first row of anchors, increases as the depth of excavation to the upper ground anchor increases. This deformation is often a significant contributor to total wall permanent deformations. Additional displacement constraints are invoked by reducing the span between the ground anchors, which will reduce the bulging deformations. The relationships developed by FHWA-RD-98-067 are recommended in a "displacement control" design procedure given in FHWA-RD-97-130 (page 147) and have been adopted for use in this report. Specifically, the FHWA-RD-97-130 Equation 9.1 is used to estimate cantilever displacement y_c, and Equation 9.2 is used to used to estimate bulging deformations y_b and will be given subsequently. The designer sets project-specific horizontal displacement limitations, which, in turn, are set as limiting values for y_c and y_b. The first-row anchor depth and spacings for the subsequent rows of anchors are then established that meet this project-specific displacement performance objective. A subsequent example calculation will demonstrate this procedure. On page 148 of FHWA-RD-97-130 the designer is cautioned that movements estimated from these two equations show trends, and they can be used to evaluate the impact of different ground anchor locations. They represent minimum movements that might be expected.

The third distinguishing aspect of the "stringent displacement control" design procedure is the factor of safety used in the EPF computation, set equal to 1.5 as compared with the 1.3 value used in the "safety with economy" design procedure. For this 50-ft-high wall problem, the EPF now becomes 26.0 pcf, which is 15 percent greater than the 22.7-pcf EPF value used in Section 3.2 "safety with economy" tieback wall design. Recall that the EPF value will scale the apparent earth pressure diagram used to compute the horizontal design anchor forces, designated as variable T_i in this report (where the subscript i designates the anchorage row number). It is inferred that by using a factor of safety equal to 1.5 in the development of apparent pressure diagram, nearer to at-rest conditions (versus active earth pressure conditions) will occur behind the wall, which along with smaller distance to upper ground anchor and closer anchor spacings, will contribute to reduce wall displacements over a "safety with economy" design. When displacement control of flexible tieback walls is a key consideration, the reader is referred to helpful discussions contained within Section 9.1 of FHWA-RD-97-130; Section 2.1.3 of FHWA-RD-81-150; Section 5.11.1 in FHWA-SA-99-015. It should be recognized, however, that where displacement is important to project performance, NLFEM-SSI analysis might be required to properly assess displacement performance. Additional information on NLFEM analysis can be found in Strom and Ebeling (2001). Alternatively, stiff tieback walls should always be considered in those situations where the magnitude of flexible tieback wall deformations (cantilever, bulging, and/or cumulative/final displacements) are of concern (see Strom and Ebeling 2001 or Strom and Ebeling 2002a).

Displacement limits are project specific. For this particular 50-ft-high wall design example, a maximum lateral wall displacement of 0.5 in. will be established for the Mueller et al. (1998) cantilever displacement y_c and the bulging deformation y_b equations.

With the minimum number of four rows of anchors, the vertical anchor spacing from the "safety with economy" design is as follows:

H_1 = 7 ft, 0 in. and $H_2 = H_3 = H_4 = H_5$ = 10 ft, 9 in.

These anchor spacings will be evaluated using Equations 9.1 and 9.2 of FHWA-RD-97-130 to determine if the associated cantilever and interior spans can be used to meet stringent displacement control performance requirements.

Approximate cantilever deformation y_c allowing 1.5 ft overexcavation for placement of top anchor, $h_1 = 7 + 1.5 = 8.5$ ft and with $E_s = 2850$ psi for stiff clay, and $K_o = 0.5$,

$$y_c = \frac{4K_0 \gamma\, h_1^2}{E_s} = \frac{4 * 0.5 * 132 * 8.5^2}{2850 * 12} = 0.56 \text{ in.} > 0.5 \text{ in.} \quad \text{NG}$$

The soil modulus (E_s) was obtained from Table 20 of FHWA-RD-97-130.

Approximate span bulging deformation y_b with h = 10.75 ft and wall height = 50 ft,

$$y_b = \frac{0.8K_0 \gamma hL}{E_s} = \frac{0.8 * 0.5 * 132 * 10.75 * 50}{2850 * 12} = 0.83 \text{ in.} > 0.5 \text{ in.} \quad \text{NG}$$

Anchor spacing must be reduced to limit deformation to less than half an inch. Revise spacing using deformation constraints.

Cantilever deformation,

$$y_c = \frac{4K_0 \gamma h_1^2}{E_s} = \frac{4 * 0.5 * 132 * h_1^2}{2850 * 12} = 0.5 \text{ in.}$$

$$h_1 = 8.05 \text{ ft}$$

Allowing 1.5 ft of excavation below anchor point

$$\text{H}_1 = 8.05 - 1.5 = 6.54 \text{ ft}$$

Span bulging deformation,

$$y_b = \frac{0.8K_0 \gamma h L}{E_s} = \frac{0.8 * 0.5 * 132 * \text{h} * 50}{2850 * 12} = 0.5 \text{ in.}$$

$$h = 6.48 \text{ ft}$$

Try an eight-tier anchor system with $\text{H}_1 = 6'\text{–}3"$ and

$$H_2 = H_3 = H_4 = H_5 = H_6 = H_7 = H_8 = 6'\text{–}3"$$

Anchor spacing satisfies the cantilever and bulging deformation constraints of not greater than 0.5 in. by the Mueller et al. (1996) equations. Note that no constraints on total (i.e., post-construction) horizontal and vertical wall deformations were considered in these computations. Recall that FHWA-RD-97-130 relationships for <u>average</u> maximum horizontal displacements and <u>average</u> maximum settlement (assuming good construction practice in conjunction with good design) may be on the order of 1.2 in. and 0.9 in., respectively. For displacement-sensitive projects, NLFEM analysis of the flexible wall is recommended. Alternatively, a stiff tieback wall system may be considered.

3.3.2 Apparent earth pressure

Referring to the Section 3.2.1 calculations, the effective earth pressure (p_e) for the stringent displacement control design becomes

$$p_e = \frac{\text{Total earth pressure load } (P_{tl})}{H - \frac{H_1}{3} - \frac{H_8}{3}}$$

$$= \frac{65010}{50 - \frac{6.25}{3} - \frac{6.25}{3}} = 1418 \text{ psf}$$

3.3.3 Bending moments on soldier beams

Referring to the Section 3.2.3 calculations, the cantilever bending moment (M_1) and interior span moments (MM_1) are determined for the stringent displacement control design as follows:

$$M_1 = \frac{13}{54} H_1^2 p = \frac{13}{54} * 6.25^2 * 1418 = 13335 \text{ lb} - \text{ft/ft}$$

and,

$$\begin{aligned} MM_{(1,2,3)} &= \frac{1}{10}\left(\text{larger of } H_{(2,3,4)}\right)^2 p \\ &= \frac{1}{10}(6.25)^2(1418) \\ &= 5539 \text{ lb} - \text{ft/ft} \end{aligned}$$

hence,

Maximum moment M = 13335 lb - ft / ft (Moments are not balanced because of deformation constraints on vertical anchor spacing)

3.3.4 Subgrade reaction using tributary area method

Referring to the Section 3.2.4 calculations, the subgrade reaction (R) is determined for the stringent displacement control design as follows:

$$R = \left(\frac{3}{16} H_8\right) p \quad \text{(see Figure 5.4, Strom and Ebeling 2001)}$$

i.e.,

$$\begin{aligned} R &= \frac{3}{16} 6.25 * 1418 \\ &= 1662 \text{ lb / ft} \end{aligned}$$

3.3.5 Ground anchor load horizontal components

Referring to the Section 3.2.5 calculations the horizontal component of each tier of anchors is determined for the stringent displacement control design. Assume soldier spacing (s) = 6 ft.

Top tier:

$$T_1 = \left(\frac{2}{3}H_1 + \frac{1}{2}H_2\right)p$$

$$= \left(\frac{2}{3} * 6.25 + \frac{1}{2} * 6.25\right)1418$$

$$= 10340 \text{ lb/ft}$$

(Design anchor force = 10.34 kips/ft × 6 ft/cos 20° = 66 kips)

Tiers 2, 3, 4, 5, 6:

$$T_2 = T_3 = T_4 = T_5 = T_6 = \frac{1}{2}\left(H_2 + H_3\right)p$$

$$= \frac{1}{2}(6.25 + 6.25)\,1418$$

$$= 8863 \text{ lb / ft}$$

(Design anchor force = 8.863 kips/ft × 6 ft/cos 20° = 56.6 kips)

Lower tier:

$$T_n = \left(\frac{H_n}{2} + \frac{23}{48}H_{n+1}\right)p$$

i.e.,

$$T_7 = \left(\frac{H_7}{2} + \frac{23}{48}H_8\right)\text{p}$$

$$= \left(\frac{6.25}{2} + \frac{23 * 6.25}{48}\right)1418$$

$$= 8678 \text{ lb/ft}$$

(Design anchor force = 8.678 kips/ft × 6 ft/ cos 15° = 53.9 kips)

Use T_{max} = 10340 lb/ft   (Note: the unbalanced anchor loads are the result of vertical spacing used to satisfy deformation contraints.)

3.3.6 Soldier beam size

Assume soldier beam spacing (s) = 6.0 ft.

Note that the minimum permissible spacing is 4.0 ft (see Figure 8.5 of Strom and Ebeling 2001).

Hence, the maximum soldier beam design moment (M_{Max}) for the stringent displacement control design is

$$\text{Maximum design moment } (M_{Max}) = \frac{13{,}335}{1000} * 6 = 80.0 \text{ ft - kip.}$$

In accordance with Corps criteria (HQUSACE 1991), the allowable stresses for the soldier beams and wales shall be as follows:

Bending (i.e., combined bending and axial load): $f_b = 0.5\, f_y$

Shear: $f_v = 0.33\, f_y$

Allowable stresses are based on 5/6 of the AISC-ASD recommended values (AISC 1989) and reflect the Corps' design requirements for steel structures. Thus,

The allowable bending stress for Grade 50 steel: $F_b = 0.5\, F_y = 25$ ksi

Allowable shear stress for Grade 50 steel: $F_v = 0.33\, F_y = 16.5$ ksi

The required section modulus (S) for the stringent displacement control design using Grade 50 steel is

$$S = \frac{M_{max}}{F_b} = \frac{80 * 12}{25} = 38.4 \text{ in.}^3$$

From AISC (1989), HP 10×42 provides $S_{xx} = 43.4 > 38.4$ in.^{3} *OK*

or,

2 MC 10×22 provides $S_{xx} = 41 > 38.4$ in.^{3} *OK*

Try 2 MC 10×22 Grade 50 steel for economical section.

Check shear capacity:

Maximum shear force, $V_{Max} = T_{max} * 6 = 10{,}340*6 = 62{,}040$ lb = 62 kips

Required area, $A = 62/16.5 = 3.76$ in.^{2}

Shear area provided by 2 MC 10×22

$= 2*d*t_w = 2*10*0.290 = 5.8$ in$^2 > 3.76$ in.^{2} OK

where d and t_w are depth and width of MC 10×22.

Use 2 MC 10×22 Grade 50 section.

3.3.7 Anchor lengths

As for the "safety with economy" design, for constructibility, the upper three tiers of ground anchors will be inclined downward at an angle of 20 deg, and the lower tier inclined downward at an angle of 15 deg (see Figure 3.2).

3.3.7.1 Unbonded anchor length, L_i. Assume inclination of 20 degrees for top anchors and 15-degree inclination for bottom-tier anchor. The unbonded length must be sufficient such that anchor bond zone is beyond the short-term (undrained) and long-term (drained) potential failure surfaces and satisfy the Figure 8.5 Strom and Ebeling (2001) length criteria. With the short-term shear strength characterized in terms of S_u equal to 2,400 psf (with $\phi = 0$ deg), and with the long-term shear strength characterized in terms of ϕ' equal to 36 degrees, the short-term loading condition will require greater unbonded anchor lengths. Thus, the potential failure plane will be based on the undrained shear strength with $\phi = 0$ degree (as is also done in the FHWA-RD-97-130 cohesive design example; refer to Figure 107).

$$\frac{\text{unbonded length, } L}{\varphi} = \frac{\text{height of anchor point}}{\alpha}$$

$$\varphi = 45° - \frac{\phi}{2} = 45° - 0° = 45°$$

$$\alpha = 180° - 70° - 45° = 65°$$

Top-tier anchor:

$$\frac{L}{\sin 45} = \frac{43.75}{\sin 65}$$

$$L = \frac{43.75 * \sin 45}{\sin 65} = 34.1 \text{ ft}$$

Allowing 5 ft or 0.2H clearance beyond shear plane (see Figure 8.5, Strom & Ebeling 2001),

$$L_1 = 34.1 + (0.2H \text{ or } 5 \text{ ft whichever is greater})$$

$$= 34.1 + 10 = 44.1 \text{ ft} > 15 \text{ ft minimum required for strand anchor} \quad OK$$

(Minimum required for bar anchor is 10 ft)

Similarly, for the second-tier anchor:

$$L_2 = \frac{37.5}{43.75} * L + 0.2H$$
$$= 39.2 \text{ ft} > 15 \text{ ft OK}$$

Third-tier anchor:

$$L_3 = \frac{31.25}{43.75} * L + 0.2H$$
$$= 34.4 \text{ ft} > 15 \text{ ft OK}$$

Fourth-tier anchor:

$$L_4 = \frac{25}{43.75} * L + 0.2H$$
$$= 29.5 \text{ ft} > 15 \text{ ft OK}$$

Fifth-tier anchor:

$$L_5 = \frac{18.75}{43.75} * L + 0.2H$$
$$= 24.6 \text{ ft} > 15 \text{ ft OK}$$

Sixth-tier anchor:

$$L_6 = \frac{12.5}{43.75} * L + 0.2H$$
$$= 19.7 \text{ ft} > 15 \text{ ft OK}$$

Lower-tier anchor:

$$L_7 = \frac{6.25}{\sin 60} * \sin 45 + 0.2H$$
$$= 15.1 \text{ ft} > 15 \text{ ft OK}$$

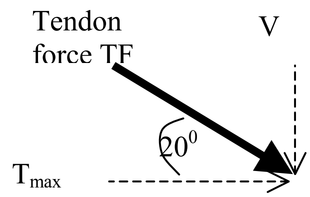
The unbonded length determined above should be verified using the internal stability analysis procedures described in Strom and Ebeling (2002b) for both short-term (undrained) and long-term (drained) conditions. The verification process uses limiting equilibrium procedures, which can be performed by simple hand calculations or GPSS procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to meet internal stability performance requirements (i.e., factor of safety of 1.5 for a stringent displacement control design).

3.3.7.2 Bonded length of anchors, L_b. A preliminary estimate of the bond length (L_b) required to develop the ground anchors is provided below. The horizontal anchor force T_1 corresponds to maximum horizontal anchor force T_{max}

(Section 3.3.5). The horizontal anchor forces T_2 through T_6 are of equal magnitude. Because the horizontal anchor forces T_2 through T_6 and anchor force T_7 are within 14 percent and 16 percent, respectively, of this T_{max} value, the bond length computations will be made using the tendon force value of T_{max}. The computed bond length will be slightly conservative for anchor tendons 2 through 7.

With 6-ft spacing between soldier beams and using T_{max} = 10,340 lb/ft, for all anchors, the maximum anchor (tendon) force TF is

$$TF = \frac{T_{max} * 6}{\cos 20°} = \frac{10340 * 6}{\cos 20°} = 66022 \text{ lb} \approx 66 \text{ kips}$$



An empirical method (Equation 1.2) was attempted to estimate the bond length of large-diameter straight-shaft gravity-grouted anchors for preliminary design purposes. Recall that it is up to the tieback anchor contractor, usually a specialty subcontractor, to propose the type of anchorage system to be used to meet the wall design requirements.

The design tendon force for the anchor bond zone is computed equal to 66 kips. Applying a factor of safety equal to 2.0 to this design force results in an ultimate anchor force equal to 132 kips.

No site-specific anchor load test data are available for use in this design. Consequently, preliminary design computations are made using traditional (non-site specific) assumptions for the range in value of the adhesion factor when computing an average ultimate soil-to-grout bond stress (Equation. 1.3) and subsequently, the ultimate capacity of a large-diameter straight-shaft gravity-grouted anchor of 40-ft length (the maximum possible length without requiring specialized methods). The ultimate anchor capacity of a large-diameter straight-shaft gravity-grouted anchor was computed using Equation. 1.2. These computations (not shown) indicate that a more robust anchorage system will be required to achieve an ultimate anchorage capacity of 132 kips.

Review of information contained in Section 1.5.4 indicates that a post-grouted (regroutable) ground anchor is a possible solution. (The anchor capacities cited for the case histories given and for the soil conditions cited in this section makes this type of anchorage system a viable candidate.) For a ultimate anchor force equal to 132 kips and assuming a 40-ft bond length, the ultimate capacity of rate of load transfer corresponds to 3.3 kips per lineal ft. A preliminary bonded length L_b of 40 ft will be assumed for anchorage layout purposes. Recognize that the final bond length, anchor capacity, etc., will be confirmed by proof-testing and performance testing onsite (Strom and Ebeling 2002b).

3.3.7.3 Total anchor lengths ($Lt_i = L_i + L_b$).

Top-tier anchor:

$$Lt_1 = 44.1 + 40 = 84.1 \text{ ft} \approx 85 \text{ ft}$$

Second-tier anchor:

$$Lt_2 = 39.2 + 40 = 79.2 \text{ ft} \approx 80 \text{ ft}$$

Third-tier anchor:

$$Lt_3 = 34.4 + 40 = 74.4 \text{ ft} \approx 75 \text{ ft}$$

Fourth-tier anchor:

$$Lt_4 = 29.5 + 40 = 69.5 \text{ ft} \approx 70 \text{ ft}$$

Fifth-tier anchor:

$$Lt_5 = 24.6 + 40 = 64.6 \text{ ft} \approx 65 \text{ ft}$$

Sixth-tier anchor:

$$Lt_6 = 19.7 + 40 = 59.7 \text{ ft} \approx 60 \text{ ft}$$

Lower-tier anchor:

$$Lt_7 = 15.1 + 40 = 55.1 \text{ ft} \approx 56 \text{ ft}$$

The total anchor length (bonded + unbonded length) determined above should be verified using the external stability analysis procedures described in Strom and Ebeling (2002b) for both short-term (undrained) and long-term (drained) conditions. This verification process also uses limiting equilibrium procedures, which can be performed by simple hand calculations or GPSS procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to prevent ground mass stability failure (i.e., meet external stability performance requirements with factor of safety of 1.5 for a stringent displacement control design).

3.3.8 Anchor strands

The number of 0.6-in.-diam ASTM A416, Grade 270, strands (ASTM 1999) required to meet stringent displacement control design requirements is determined. It is assumed that the final design force after losses will be based on an allowable anchor stress of 0.6 f_y, or 35.2 kips per strand.

Use the same maximum anchor load TF = 66 kips for sizing all seven anchor strands (since $T_1 = T_{max}$, and T_2 through T_6 are within 14 percent of T_{max} and T_7 is within 16 percent of T_{max}).

From Table 8.5, Strom and Ebeling (2001),

Capacity of two 0.6-in. strands = 70.4 kips > 66 kips OK

Use two 0.6-in. strands.

3.3.9 Drilled-in shaft diameter

The drilled-in soldier beam will be fabricated from a pair of MC 10×22 shapes (using Grade 50 steel), as discussed in Section 3.3.6. Additionally, a 12-in.-diameter hole, with casing, will be used to construct the anchor bond zone for all anchors, as discussed in Section 3.3.7.2.

The depth, d, and flange width, b_f, of an MC 10×22 are

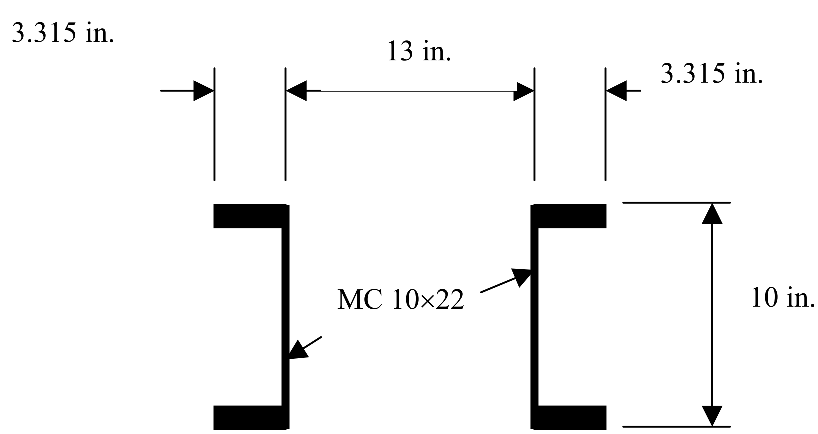
d = 10 in.

and

b_f = 3.315 in.

From Table 8.6, Strom and Ebeling (2001), trumpet diameter for three 0.6-in. strands and Case I corrosion protection = 5-7/8 in.

The distance between channels required is 13 in. to allow ample room for the installation of the anchor zone. For anchor zone details see Figure 10.2(b) Strom and Ebeling (2001).



The diameter for the drilled shaft (b) required to install the soldier beams is determined next.

For the previously described configuration of the pair of MC 10×22 shapes, the diagonal (from flange tip to flange tip) is given by

$$diagonal = \sqrt{d^2 + (2b_f + \text{clear spacing})^2}$$

$$diagonal = \sqrt{(10)^2 + (2 \bullet 3.315 + 13)^2}$$

$$diagonal = \sqrt{(10)^2 + (19.63)^2}$$

$$diagonal = 22.03 \text{ in.}$$

To install the fabricated pair of MC 10×22 shapes, the diameter of the drilled shaft (b) must be greater than the flange tip to flange tip diagonal of 22.03 in. Use 26-in.-diameter drilled shaft.

3.3.10 Temporary timber lagging

Lagging selection for the stringent displacement control design is identical to that indicated for the "safety with economy" design (Section 3.2.10).

3.3.11 Soldier beam toe embedment

As with the "safety with economy" design, soldier beam toe embedment requirements for both vertical and horizontal loads must be determined. For the "stringent displacement control" design, with respect to the vertical component of prestress anchor load:

Vertical component of anchor load:

$$\begin{aligned}\sum V &= \left(V_1 + V_2 + V_3 + V_4 + V_5 + V_6 + V_7\right) * 6 \\ &= \left[\left(T_1 + T_2 + T_3 + T_4 + T_5 + T_6\right) * \tan 20° + T_7 * \tan 15°\right] * 6 \\ &= \left[\left(10340 + 5 * 8863\right) * \tan 20° + 8678 * \tan 15°\right] * 6 \\ &= 133{,}308 \text{ lb} = 133.3 \text{ kips}\end{aligned}$$

FHWA-SA-99-015 (page 95) general design recommendations for concrete backfill of predrilled holes include the use of structural concrete from the bottom of the hole to the excavation base and a lean-mix concrete for the remainder of the hole. The design concept is to provide maximum strength and load transfer in the permanently embedded portion of the soldier beam while providing a weak concrete fill in the upper portion, which can be easily removed and shaped to allow lagging installation. However, contractors often propose the use of lean-mix concrete backfill for the full depth of the hole to avoid the delays associated with providing two types of concrete in relatively small quantities. This design example follows the cohesive soil (stiff clay) design examples given in Section 10.2 of FHWA-RD-97-130 and in Example 2 of Appendix A of FHWA-SA-99-015, assuming that lean-mix concrete is used for the full depth of the hole.

The following computations are made to determine total force that the drilled-in shaft foundation must resist. A 13-ft depth of penetration is assumed in these computations for a 50-ft exposed wall height.

The total drilled-in soldier beam shaft weight assuming a 13-ft toe length is equal to the vertical component of anchor force plus the weight of a pair of MC 10×22 plus the weight of lean-mix concrete backfill plus the weight of timber lagging. The axial load transfer from the drilled shaft above the bottom of the wall to the retained soil, which acts upward on the soldier pile, is also included in the computations. The magnitude of each of these forces is summarized in the following six steps:

a. Vertical component of anchor force = 133.3 kips.

b. Computation of the axial load transfer from the drilled shaft above the bottom of the wall to the retained soil:

(1) Axial load and ground movements are interrelated. The magnitude of the axial load depends upon the vertical components of the ground anchor loads, the strength of the supported ground, vertical and lateral movements of the wall, the relative movements of the ground with respect to the wall, and the axial load carrying capacity of the toe. FHWA-RD-98-066 (page 66) discusses results taken from walls in dense sands and stiff to hard clays in which the axial load measured in the soldier beam toes was <u>less</u> than the vertical components of the ground anchors. This favorable axial load transfer from the soldier beam to the retained soil is idealized in Figure 41(b) of FHWA-RD-97-130. Axial load transferred to the ground above the bottom of the excavations in stiff to hard clays was equal to A_s times ($0.25S_u$) according to FHWA-RD-97-130 (page 88). A_s was the surface area of the soldier beam in contact with the ground above the bottom of the excavation, and $0.25S_u$ was the back-calculated adhesion. At the stiff cohesive sites, the load transferred from the soldier beam to the ground above the bottom of the excavation appears to be valid for the long-term condition. Using an adhesion equal to 25 percent of the undrained shear strength gives a lower load transfer rate than a rate based on drained shear strengths.

(2) To take advantage of this axial load transfer from the soldier beam to the retained stiff to hard (cohesive) soil, Table 11 in FHWA-RD-98-066 and Table 11 in FHWA-RD-97-130 stipulate that

$$S_u > \frac{\gamma \bullet H}{4} - 5.714 \bullet H$$

which for this problem becomes

$$2{,}400 \text{ psf} > \frac{132 \text{ pcf} \bullet 50 \text{ ft}}{4} - 5.714 \bullet 50 \text{ ft}$$

$$2{,}400 > 1{,}650 - 285.7$$

$$2{,}400 > 1{,}364.3 \qquad OK$$

(3) So the following set of computations assume that the axial load is transferred to the ground above the bottom of the excavation in this stiff clay site. Page 209 in FHWA-RD-97-130 gives this transfer force as

$$\text{Axial load transfer} = \alpha \bullet S_u \bullet A_s \bullet \left(H - H_1\right)$$

where H is the height of the wall (= 50 ft) and H_1 is the depth to the first row of anchors (6.25 ft in Section 3.3.1). A_s is approximated as equal to half the circumference of the drilled-in soldier beam shaft,

$$A_s = \frac{1}{2} \bullet \pi \bullet d_s$$

With d_s equal to 26 in. (Section 3.3.9), A_s equals 40.84 in. (3.403 ft).

$$\text{Axial load transfer} = 0.25 \bullet 2400 \text{ psf} \bullet 3.403 \text{ ft} \bullet \left(50 \text{ ft} - 6.25 \text{ ft}\right)$$

$$= 600 \text{ psf} \bullet 3.403 \text{ ft} \bullet 43.75 \text{ ft} = 89{,}329 \text{ lb} = 89.3 \text{ kips}$$

(4) Note that this 89.3-kip force acts to reduce the axial load acting on the soldier beam foundation. The magnitude of this upward-acting force (from the perspective of the soldier beam) is significant. Great care must be exercised by the designers when taking advantage of this load transfer mechanism. It is assumed in this wall design that the soldier beam wall settles relative to the ground. Before applying this force in a design, designers should review the discussion and guidance given on pages 87-90 of FHWA-RD-97-130 and pages 66-69 in FHWA-RD-98-066. The instrumented wall case histories are discussed in FHWA-RD-98-066.

c. Weight of 2 MC 10×22 channels for 56-ft length = 2*0.022*63 = 2.77 kips

d. Compute the weight of lean-mix concrete backfill for a drilled-in soldier beam of length 63 ft:

(1) Weight of lean-mix concrete backfill for a drilled shaft diameter (d_s) of 26 in. and a drilled-in soldier beam cylinder of length 63 ft.

$$\text{Total area} = \pi \bullet \frac{d_s^{\,2}}{4}$$

$$\text{Total area} = \pi \bullet \frac{(26)^2}{4}$$

$$\text{Total area} = 530.93\ \text{in}^2 = 3.687\ \text{ft}^2$$

$$\text{Gross weight} = \left(0.145\frac{\text{kips}}{\text{ft}^3}\right) \bullet \text{Total area} \bullet 63\ \text{ft}$$

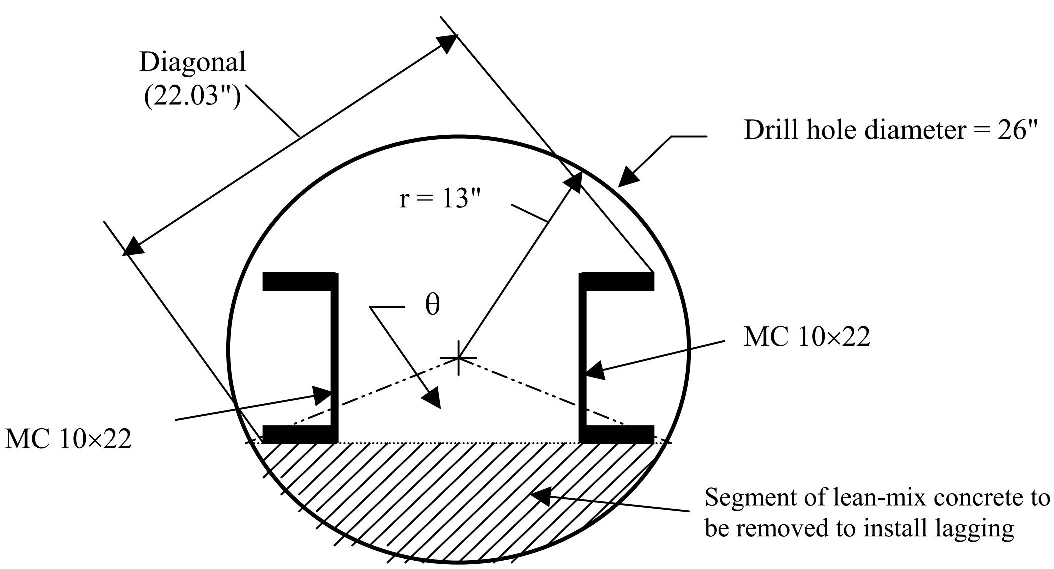
$$\text{Gross weight} = \left(0.145\frac{\text{kips}}{\text{ft}^3}\right) \bullet (3.687) \bullet 63$$

Gross weight = 33.68 kips

That is, the gross weight of a 63-ft-high, 26-in.-diameter lean-mix concrete cylinder is 33.68 kips. (This does not account for the weight of lean-mix concrete removed when placing the lagging.)

(2) Reduction in gross weight of a 63-ft-long cylinder for removal of the lean-mix concrete backfill during lagging installation.

- Compute the area of lean-mix concrete to be removed down the 50 ft (exposed) of height in front of the flanges of the pair of MC 10×22 shapes:



- Compute the segment (of circle) area in front of flanges to be removed:

$$\theta = 2 \bullet \left[\cos^{-1}\left(\frac{\text{half channel depth}}{\text{radius}} \right) \right] = 2 \bullet \left[\cos^{-1}\left(\frac{5}{13} \right) \right]$$

$$= 134.76 \text{ deg}$$

where

$$\text{half channel depth} = \frac{d}{2} = \frac{10}{2} = 5 \text{ in.}$$

$$r = \text{radius} = \frac{\text{diameter}}{2} = \frac{d_s}{2} = \frac{26}{2} = 13 \text{ in.}$$

$$\text{Segment area} = \pi r^2 \frac{\theta}{360} - r^2 \frac{\sin\theta}{2} = 138.75 \text{ in.}^2 = 0.96 \text{ ft}^2$$

The exposed wall height equals 50 ft. The area of lean-mix concrete to be removed down the 50 ft of exposed wall in front of the flanges for the pair of MC 10×22 shapes is equal to 0.963 ft^2 per ft of exposed wall height. For this 26-in.-diameter drilled-in shaft, the area removed represents approximately 26 percent of the cross-sectional area of the original 26-in.-diameter (cylinder) of lean-mix concrete per ft of exposed height.

- Compute the weight of lean-mix concrete removed during placement of lagging over the 50 ft of exposed height of wall:

$$\text{Weight removed} = \left(0.145 \frac{\text{kips}}{\text{ft}^3} \right) \bullet \text{Segment area} \bullet \text{H}$$

$$\text{Weight removed} = \left(0.145 \frac{\text{kips}}{\text{ft}^3} \right) \bullet 0.963 \text{ ft}^2 \bullet 50 \text{ ft} = 6.98 \text{ kips}$$

(3) Compute the weight of lean-mix concrete backfill for a drilled-in soldier beam of length 63 ft less the weight removed during placement of lagging:

Lean-mix net weight = Gross weight – Weight removed

Lean-mix net weight = 33.68 - 6.98 = 26.7 kips *e.* Compute the weight of timber lagging over 50 ft exposed height for a span of 6 ft:

$$\text{Lagging weight} = \left(0.05\,\frac{\text{kips}}{\text{ft}^3}\right) \bullet \text{span} \bullet \text{height} \bullet \text{thickness}$$

$$\text{Lagging weight} = \left(0.05\,\frac{\text{kips}}{\text{ft}^3}\right) \bullet 6\text{ ft} \bullet 50\text{ ft} \bullet \left(\frac{3}{12}\right) = 3.75\text{ kips}$$

f. Compute the applied total drilled-in soldier beam axial load:

$$Q_{applied} = \sum V \text{ - Axial load transfer + Weight of channels +}$$
$$\text{Lean mix net weight + Lagging weight}$$

$$Q_{applied} = 133.3\text{ kips - }89.3\text{ kips} + 2.77\text{ kips} + 26.7\text{ kips} + 3.75\text{ kips}$$
$$= 77.2\text{ kips}$$

Thus, for the 26-in.-diameter drilled-in shaft with a 6-ft depth of penetration, the applied axial load is equal to 77.2 kips.

3.3.12 Depth of toe penetration, D

This section outlines the depth of penetration computations. For a drilled-in shaft,

Ultimate axial resistance = skin friction resistance + tip resistance

Hence:

$$Q_{ult} = Q_{skin} + Q_{tip} \qquad \text{(see Equation 8.25 in Strom and Ebeling 2001)}$$

The factors of safety for axial capacity of drilled-in soldier beams in cohesive soil are

$FS_{skin} = 2.5$ and $FS_{skin} = 2.5$

according to Table 8.9 in Ebeling and Strom (2001) and Table 8.9 in FHWA-SA-99-015. Thus, the allowable axial load Q_{all} is given by

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} \left(\begin{array}{l}\text{modified form of Equations 8.18} \\ \text{in Strom and Ebeling 2001}\end{array}\right)$$

The traditional potential foundation failure mode due to axial loading is based on the assumption of the drilled-in shaft being fully effective in transferring the applied vertical load from the pair of steel channels through the lean-mix concrete mix to the surrounding soil. The corresponding traditional computation assumes the axial capacity is due to the drilled-in shaft acting as a single structural unit within the surrounding granular soil media. FHWA-SA-99-015 (page 95) and FHWA-RD-97-130 (page 90) note that for lean-mix backfilled drilled-in shafts, the lean-mix concrete may not be sufficiently strong to allow vertical load transfer from the soldier beam to the concrete. Consequently, a second potential failure mode must also be considered: the alternative potential failure mode assumes the soldier beam "punches" through the lean-mix, in which case the drilled-in shaft cross-section (assumed to be rectangular) will not be effective in transferring the load to the surrounding soil. Both potential failure modes are evaluated and the smallest capacity is used in the design. These computations are demonstrated in the following two sets of analyses. Note that in each set of computations a different value for depth of penetration D was used. Since the mechanisms for the two potential modes of failures are different, the minimum depth of penetration values required to satisfy the factors of safety against failure will also differ.

3.3.12.1 Analysis 1: Drilled-in shaft capacity (with shaft acting as single unit). In Section 3.3.9 the diameter of the drilled-in shaft was established to be 26 in. By trial and error using the following design analysis procedure, it is determined that a depth of penetration (D) equal to 13 ft is required to meet the aforementioned factor of safety requirements. The soldier beam length is equal to 63 ft (= H + D = 50 ft + 13 ft).

a. *Ultimate skin friction.* The ultimate resistance due to skin friction, Q_{skin}, is given by

$$Q_{skin} = f_{skin} \bullet A_{cylinder}$$

The average unit skin friction is computed using the Strom and Ebeling (2001) Equation 8.28 to be

$$f_{skin} = \alpha \bullet S_u \quad \text{with the limitation that } f_{skin} < 5.5 \text{ ksf}$$

where α is equal to 0.55. Thus,

$$f_{skin} = 0.55 \bullet 2.4 \text{ ksf} = 1.32 \text{ ksf}$$

The surface area of the drilled-in shaft is given by

$$A_{cylinder} = \pi \bullet (\text{diameter}) \bullet D$$

$$A_{cylinder} = \pi \bullet \left[26 \text{ in.} \bullet \left(\frac{\text{ft}}{12 \text{ in.}} \right) \right] \bullet 13 \text{ ft} = 88.488 \text{ ft}^2$$

Thus, the ultimate resistance due to skin friction along the 6-ft-long depth of penetration of the drilled-in shaft is

$$Q_{skin} = f_{skin} \bullet A_{cylinder} = 1.32 \bullet 88.488 = 116.8 \text{ kips}$$

b. Ultimate tip resistance. The ultimate tip resistance due to end bearing, Q_{tip}, is given by

$$Q_{tip} = q_b \bullet A_{tip}$$

The unit end bearing ultimate resistance is computed using the Strom and Ebeling (2001) Equation 8.29 as

$q_b = N_c \bullet S_u$ with the limitation that $q_b < 80$ ksf

and

$N_c = 6 \bullet \left[1 + 0.2 \bullet \left(\frac{D}{\text{diameter}}\right)\right]$ with the limitation that $N_c < 9$

For the assumed depth D = 13 ft,

$$N_c = 6 \bullet \left[1 + 0.2 \bullet \left(\frac{13}{26/12}\right)\right] = 13.2$$

Use N_c equal to 9.

$$q_b = 9 \bullet 2.4 \text{ ksf} = 21.6 \text{ ksf}$$

The cross-sectional area of the tip is

$$A_{tip} = \pi \bullet \frac{(\text{diameter})^2}{4}$$

$$A_{tip} = \pi \bullet \frac{\left[26 \text{ in.} \bullet \left(\frac{\text{ft}}{12 \text{ in.}}\right)\right]^2}{4} = 3.687 \text{ ft}^2$$

Thus, the ultimate tip resistance of the 26-in.-diameter drilled-in shaft is

$$Q_{tip} = q_b \bullet A_{tip} = 21.6 \bullet 3.687 = 79.6 \text{ ksf}$$

c. *Ultimate axial load resistance.* The ultimate axial load Q_{ult} is computed to be

$$Q_{ult} = Q_{skin} + Q_{tip} = 116.8 + 79.6 = 196.4 \text{ kips}$$

Note that skin friction provides 60 percent of the ultimate axial load resistance, while tip resistance due to end bearing provides 40 percent of this ultimate axial load value.

d. *Allowable axial load.* The allowable axial load Q_{all} is computed to be

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} = \frac{116.8}{2.5} + \frac{79.6}{2.5} = 46.7 + 31.8 = 78.5 \text{ kips}$$

Note that skin friction provides 60 percent of the allowable axial load resistance, while tip resistance due to end bearing provides 40 percent of this allowable axial load value.

The allowable axial load Q_{all} for this 26-in.-diameter drilled-in shaft with an assumed 13-ft depth of embedment is 78.5 kips, which is 1.3 kips larger than the applied axial load of 77.2 kips (see Section 3.3.11), i.e., $Q_{applied} > Q_{all}$. Thus, a 13-ft depth of penetration is acceptable for this assumed potential mode of foundation failure. Recall that this potential mode of foundation failure assumes the shaft acting as single unit.

Solution process: The procedure used to determine the depth of penetration D in this problem was as follows: assume a depth of penetration; compute the ultimate axial load resistance Q_{ult}; compute the allowable axial load Q_{all}; compute the total applied axial load for the drilled-in soldier beam system $Q_{applied}$ (Section 3.3.11); adjust the depth of penetration D as necessary; and repeat computations until Q_{all} is approximately equal to $Q_{applied}$. Ensure that for the final value of D used in the computations Q_{all} is greater than or equal to $Q_{applied}$.

3.3.12.2 Analysis 2: "Punching" soldier beams capacity. The drilled-in soldier beam backfilled with a lean-mix concrete has the potential not to act as a single structural unit in which the axial load is transferred from the pair of channels through the lean-mix to the surrounding soil foundation but where the pair of channels "punches" through the lean-mix concrete backfill. In Section 3.3.9 the diameter of the drilled-in shaft was established to be 26 in. By trial and error using the following design analysis procedure it is determined that a depth of penetration (D) equal to 6 ft is required to meet established factor of safety requirements. (Note that this value of D differs from the 13-ft value used in Analysis 1 computations. The authors of this report are demonstrating that the minimum required depth of penetration is not the same for the two different types of failure modes.) For the Analysis 2 procedure the soldier beam length is equal to 56 ft (= H + D = 50 ft + 6 ft). As an alternative the designer could verify that the depth of penetration established by the design Analysis 1 procedure satisfies factor of safety requirements for the Analysis 2 procedure.

The following computations assume the pair of soldier beam channels will punch through the lean-mix concrete backfill rather than transfer the load through the backfill to the ground. Note that in these computations the diameter of the drilled shaft is not used (i.e., the drilled shaft cross-section will not be effective in transferring the load to the surrounding soil). Instead, the rectangular "box" perimeter of the pair of channels is used in both the skin friction and tip resistance computations.

a. *Ultimate skin friction.* The ultimate resistance due to skin friction, Q_{skin}, is given by

$$Q_{skin} = f_{skin} \bullet A_{box}$$

The average unit skin friction for this "punching" mode of failure is computed using the Strom and Ebeling (2001) Equation 8.20 to be

$$f_{skin} = K \bullet \sigma'_{ave} \bullet \tan(\delta)$$

with

$$\sigma'_{ave} = \gamma \bullet \left(\frac{H + D}{2} \right) = 132 \bullet \left(\frac{50 + 6}{2} \right) = 3696 \text{ psf}$$

FHWA-RD-97-130 (page 94) and FHWA-SA-99-015 (page 95) note that when a lean-mix concrete backfill is used in a drilled-in shaft, f_{skin} is computed using $K = 2$ and $\delta = 35$ degrees in the f_{skin} equation (see page 180 in Strom and Ebeling 2001). Note that these parameter values are specific to the "punching" mode of failure through the lean-mix concrete. Thus, f_{skin} becomes

$$f_{skin} = 2 \bullet \left[3696 \text{ psf} \bullet \left(\frac{\text{kips}}{1000 \text{ lb}} \right) \right] \bullet \tan(35) = 5.176 \text{ ksf}$$

The surface area of the rectangular "box" defined by the perimeter of the pair of channels is given by

$$A_{box} = [2 \bullet (\text{channel depth}) + 2 \bullet (\text{flange - to - flange width})] \bullet D$$

$$A_{box} = \left[2 \bullet (\text{channel depth}) + 2 \bullet \begin{pmatrix} 2 \bullet b_f + \text{clear space} \\ \text{between channels} \end{pmatrix} \right] \bullet D$$

$$A_{box} = [2 \bullet (10\text{ in.}) + 2 \bullet (2 \bullet 3.315\text{ in.} + 13\text{ in.})] \bullet \left(\frac{1}{12}\right) \bullet 6\text{ ft}$$

$$= [(20\text{ in.}) + (39.26\text{ in.})] \bullet \left(\frac{1}{12}\right) \bullet 6\text{ ft} = 29.63\text{ ft}^2$$

Thus, the ultimate resistance due to skin friction along the 6-ft-long depth of penetration of the drilled-in shaft is

$$Q_{skin} = f_{skin} \bullet A_{box} = 5.176\text{ ksf} \bullet 29.63 = 153.36\text{ kips}$$

b. *Ultimate tip resistance.* The ultimate tip resistance due to end bearing, Q_{tip}, is given by

$$Q_{tip} = q_b \bullet A_{tip}$$

The unit end bearing ultimate resistance is computed using the Strom and Ebeling (2001) Equation 8.29 relationship

$$q_b = N_c \bullet S_u$$

where for a rectangle

$$N_c = 5 \bullet \left\{1 + 0.2 \bullet \left[\frac{D}{(2 \bullet b_f + \text{clear spacing})/12}\right]\right\} \bullet \left[1 + 0.2 \bullet \left(\frac{D}{d}\right)\right]$$

with the limitation that N_c is less than 7.5 to 9. Recall from Section 3.2.9 that b_f is 3.405 in., clear spacing is 13 in. and d is 10 in.

$$N_c = 5 \bullet \left\{1 + 0.2 \bullet \left[\frac{6}{(2 \bullet 3.315 + 13)/12}\right]\right\} \bullet \left[1 + 0.2 \bullet \left(\frac{6}{10/12}\right)\right]$$
$$= 21.15$$

Use N_c equal to 9.

This results in a unit end bearing ultimate resistance equal to

$$q_b = 9 \bullet 2.4\text{ ksf} = 21.6\text{ ksf}$$

The cross-sectional area of the rectangular "box" tip is

$$A_{tip} = (\text{channel depth}) \bullet (\text{flange - to - flange width})$$

$$A_{tip} = (\text{channel depth}) \bullet (2 \bullet b_f + \text{clear space between channels})$$

$$A_{tip} = [(10\text{ in.}) \bullet (2 \bullet 3.315\text{ in.} + 13\text{ in.})] = [(10\text{ in.}) \bullet (19.63\text{ in.})]$$
$$= 196.3\text{ in.}^2$$

Thus, the ultimate tip resistance of the 26-in.-diameter drilled-in shaft is

$$Q_{tip} = q_b \bullet A_{tip} = 21.6 \bullet 196.3 \bullet \left(\frac{1}{144}\right) = 29.4\text{ ksf}$$

c. *Ultimate axial load resistance.* The ultimate axial load Q_{ult} is computed to be

$$Q_{ult} = Q_{skin} + Q_{tip} = 153.36 + 29.4 = 182.8\text{ kips}$$

Note that skin friction provides 84 percent of the ultimate axial load resistance, while tip resistance due to end bearing provides 16 percent of this ultimate axial load value.

d. *Allowable axial load.* The allowable axial load Q_{all} is computed to be

$$Q_{all} = \frac{Q_{skin}}{FS_{skin}} + \frac{Q_{tip}}{FS_{tip}} = \frac{153.36}{2.5} + \frac{29.4}{2.5} = 61.3 + 11.8 = 73.1\text{ kips}$$

Note that skin friction provides 84 percent of the allowable axial load resistance, while tip resistance due to end bearing provides 16 percent of this allowable axial load value.

The allowable axial load Q_{all} for this 26-in.-diameter drilled-in shaft with an assumed 6-ft depth of embedment is 73.1 kips, which is equal to the applied axial load of 73.1 kips (computations not shown but follow those made in Section 3.3.11 using a 6-ft depth of penetration). Thus, a 6-ft depth of penetration is acceptable for this assumed potential mode of foundation failure. Recall that this potential mode of foundation failure assumes the soldier beam "punches" through the lean-mix concrete backfill.

Solution process: The procedure used to determine the depth of penetration D in this problem was as follows: assume a depth of penetration; compute the ultimate axial load resistance Q_{ult}; compute the allowable axial load Q_{all}; compute the total applied axial load for the drilled-in soldier beam system $Q_{applied}$ (following the procedure outlined in Section 3.3.11); adjust the depth of penetration D as necessary; and repeat computations until Q_{all} is approximately equal to $Q_{applied}$. Ensure that for the final value of D used in the computations Q_{all} is greater than or equal to $Q_{applied}$.

3.3.12.3 Concluding remarks. The minimum required depths of penetration were computed in this section for two potential failure modes. It was found in design Analysis 1 that a 13-ft minimum depth of penetration is required to be safe by the traditional potential foundation failure in which the drilled-in shaft acts as a single structural unit within the surrounding soil media. It was found that in design Analysis 2 that a 6-ft minimum depth of penetration is required for the system to be safe against the alternative potential failure mode, which assumes the soldier beam "punches" through the lean-mix. Therefore, the required depth of penetration D for this drilled-in shaft retaining structure is 13 ft for axial load considerations. Note that a significant percentage of the axial capacity is being carried by end bearing in Analysis 1. Chapter 6 of FHWA-RD-97-130 (or Chapter 8 in Strom and Ebeling 2001) should be reviewed prior to finalizing the depth of penetration D at 13 ft for axial loading in light of this observation.

3.3.13 Lateral capacity of soldier beam toe

Assume, based on vertical load requirements, the final toe penetration (D) is 13 ft.

Check lateral capacity of soldier beam toe:

Subgrade reaction R = 1,662 lb/ft (Section 3.3.4)

Total toe reaction = 1662*6 = 9,972 lb = 9.97 kips/soldier beam

A spreadsheet incorporating the Wang-Reese passive resistance equations (Table 3.4) is used to determine lateral resistance of the soldier beam toe following the procedure outlined in Section 8.7 of Strom and Ebeling (2001) or Section 6.2 in FHWA-RD-97-130. Note that the soldier beam width of 1.636 ft (19.63 in.) is used for drilled shafts backfilled with lean concrete as per FHWA-RD-97-130 (page 111) recommendations. If structural concrete is used to backfill the shaft, then the drilled shaft diameter (26 in.) would be used in the computations. Alpha and beta in this table are angles used to define the three-dimensional geometrical configuration of the "passive" failure wedge developing in front of the soldier beam on the excavated side (refer to Figure 8.6 in Strom and Ebeling 2001). The Wang-Reese definition for β is

$$\beta = 45 + \frac{\phi'}{2}$$

With undrained conditions (i.e., short-term load case) within the cohesive soil, β is equal to 45 degrees and α is set equal to 0 degree.

Sc in this table is the clear span between piles. Sc is 4.364 ft, equal to the span (s) of 6 ft minus the soldier beam width of 1.636 ft (19.63 in.).

Passive resistance back-calculated for soldier beam and lagging systems compares favorably with passive resistance equations developed by Wang and

Table 3.4
Spreadsheet for Computing Passive Resistance for Clay ("Stringent Displacement Control" Design)

INPUT VARIABLES

unit weight	wall height	beam width	beam spacing	Su	Sc	depth disturbance	toe depth	toe reaction
0.132	50	1.636	6	2.4	4.3640	0	13	9.97

SPREADSHEET FOR EVALUATING TOE CAPACITY FOR CLAY ("stringent displacement control" design)

Toe depth,D (ft)	*(Eq 6.20) Scr (ft)*	(Eq 6.19) *single beam wedge resistance (kip/ft)*	(Eq 6.21) *row beams wedge resistance (kip/ft)*	*critical wedge resistance (kip/ft)*	(Eq 6.23) *single beam flow resistance (kip/ft)*	*critical wedge/flow resistance (kip/ft)*	(Eq 6.24) *Rankine passive resistance (kip/ft)*	*passive resistance (kip/ft)*	*Failure Mode*	*total passive force at given toe depth (kips)*	*allowance for disturbance (Eq 6.12) for depth*	*net passive force at given depth*	*factor of safety*
col 1	***col 2***	***col 3***	***col 4***	***col 5***	***col 6***	***col 7***	***col 8***	***col 9***	***col 10***	***col 11***	***col 12***	***col13***	***col 14***
0	0.0000	7.8528	39.2736	7.8528	43.1904	7.8528	28.8000	7.8528	Wedge	0.0000	0.0000	0.0000	0.0000
1	0.4671	14.8608	40.0656	14.8608	43.1904	14.8608	29.5920	14.8608	Wedge	11.3568	0.0000	11.3568	1.1391
2	0.9257	21.8687	40.8576	21.8687	43.1904	21.8687	30.3840	21.8687	Wedge	29.7215	0.0000	29.7215	2.9811
3	1.3762	28.8767	41.6496	28.8767	43.1904	28.8767	31.1760	28.8767	Wedge	55.0942	0.0000	55.0942	5.5260
4	1.8186	35.8846	42.4416	35.8846	43.1904	35.8846	31.9680	31.9680	Wedge	85.5165	0.0000	85.5165	8.5774
5	2.2534	42.8926	43.2336	42.8926	43.1904	42.8926	32.7600	32.7600	Wedge	117.8805	0.0000	117.8805	11.8235
6	2.6806	49.9005	44.0256	44.0256	43.1904	43.1904	33.5520	33.5520	Wedge	151.0365	0.0000	151.0365	15.1491
7	3.1004	56.9085	44.8176	44.8176	43.1904	43.1904	34.3440	34.3440	Wedge	184.9845	0.0000	184.9845	18.5541
8	3.5130	63.9164	45.6096	45.6096	43.1904	43.1904	35.1360	35.1360	Wedge	219.7245	0.0000	219.7245	22.0386
9	3.9187	70.9244	46.4016	46.4016	43.1904	43.1904	35.9280	35.9280	Wedge	255.2565	0.0000	255.2565	25.6025
10	4.3176	77.9323	47.1936	47.1936	43.1904	43.1904	36.7200	36.7200	Wedge	291.5805	0.0000	291.5805	29.2458
11	4.7098	84.9403	47.9856	47.9856	43.1904	43.1904	37.5120	37.5120	Wedge	328.6965	0.0000	328.6965	32.9686
12	5.0955	91.9482	48.7776	48.7776	43.1904	43.1904	38.3040	38.3040	Wedge	366.6045	0.0000	366.6045	36.7708
13	**5.4749**	**98.9562**	**49.5696**	**49.5696**	**43.1904**	**43.1904**	**39.0960**	**39.0960**	**Wedge**	**405.3045**	**0.0000**	**405.3045**	**40.6524**
14	5.8482	105.9641	50.3616	50.3616	43.1904	43.1904	39.8880	39.8880	Flow	444.7965	0.0000	444.7965	44.6135
15	6.2154	112.9721	51.1536	51.1536	43.1904	43.1904	40.6800	40.6800	Flow	485.0805	0.0000	485.0805	48.6540

Reese (1986). Several passive failure mechanisms must be evaluated for each increment of soldier beam embedment, and the pressure associated with the governing failure condition summed over each increment of depth to determine the soldier beam total passive resistance. The failure mechanism evaluation and summing process is provided in Table 3.4 for the stringent displacement control design. In Table 3.4, the pressures attributed to the various failure mechanisms are provided in columns 5 through 8, and the pressures associated with the governing failure condition are indicated in column 9. The process used for the stringent displacement control design is similar to that used for the "safety with economy" design. The equation numbers referenced in the various columns of Table 3.4 refer to equations taken from FHWA-RD-97-130. Similar equations can be found in FHWA-SA-99-015 and Strom and Ebeling (2001). Table 3.2 (presented in Section 3.2.12 for the "safety with economy" design discussion) gives the reference equation numbers associated with each of these three references.

The computations summarized in Table 3.4 are for the 50-ft-high tieback wall in stiff clay. These computations explicitly follow those given in the Figure 113 spreadsheet procedure of FHWA-RD-97-130 (page 212). The soil properties (S_u = 2,400 psf, γ = 132 psf) used for the 50-ft-high wall are the same as those of FHWA-RD-97-130. The differences in the results (that is, between Table 3.4 and Figure 113 of FHWA-RD-97-130) are due to the soldier beam width (1.636 ft in Table 3.4 versus 1.778 ft in FHWA Figure 113). In accordance with the FHWA report, Table 3.4 does not include the total active force reduction used in the granular soil examples. On page 109 of FHWA-RD-97-130, it is stated that "the Wang and Reese equations for clays do not include an active pressure term. In stiff clays the active pressure may be negative behind the wall. Considering negative pressures during design is not reasonable since the soldier beam will move away from the soil." Further, "a continuous wall will normally be used when active pressures are positive."

Table 3.4 shows that the soldier beams, 6 ft on centers with a toe penetration (D) of 13 ft, have a lateral resistance of 405.3 kips, and the factor of safety equals 40.7. In this design problem, the depth of penetration is controlled by axial load considerations.

As stated previously, Table 3.4 (as per Figure 133 of FHWA-RD-97-130) does not include computations for "total active force reduction." These computations are performed below using FHWA Equation 6.25. For tall soldier beam walls, the computations will result in a somewhat lower net passive force and lower factor of safety.

$$P_{active} = \gamma_{ave}(H + D) - 2S_u \qquad \text{Equation 6.25, FHWA-RD-97-130}$$

At the elevation corresponding to bottom of the excavation where the depth of penetration (D) is equal to zero, the active earth pressure (behind the soldier beam and below the retained side soil) is

$$P_{D=0} = \left[132(50+0) - 2(2400)\right]\frac{1}{1000} = 1.80 \text{ ksf}$$

At the toe of the soldier beam where the depth of penetration (D) is equal to 13 ft, the active pressure is

$$P_{D=13} = \left[132(50+13) - 2(2400)\right]\frac{1}{1000} = 3.516 \text{ ksf}$$

The total active force reduction (P_{AFR}) for soldier beams with a width (b) of 1.636 ft and a depth of embedment (D) of 13 ft is

$$P_{AFR} = (1.636)(13)(1.80 + 3.516)(0.5) = 56.53 \text{ kips per soldier beam}$$

Therefore, for a depth of penetration equal to 13 ft (column 1, Table 3.4), the net passive resistance (column 13, Table 3.4) is 405.3 - 56.53 = 348.77 kips, which reduces the factor of safety from 40.7 (column 14, Table 3.4) to 35. Since this factor of safety is still greater than 2.0, it can be assumed the lateral capacity of the soldier beam toe is more than adequate for a stringent displacement control design.

The authors of this report recommend that designers always consider positive active earth pressures and the effect they have in reducing net toe resistance. Recall that the focus of this report is tall tieback walls. In general, the taller the wall, the more likely it is that positive active earth pressures may be encountered in stiff clays. For this particular clay site, assuming a penetration depth of 13 ft, positive active earth pressure will begin to occur when the wall height reaches [2 (2,400)/132] - 13 = 23.36 ft. The designer should also consider the cautionary advice provided in FHWA-RD-97-130 with respect to the use of soldier beam systems under “positive active earth pressure conditions.”

A summary of the results for the “stringent displacement control” design is provided in Table 3.5.

3.3.14 Basal stability

The apparent pressure diagram used to compute the horizontal components of the anchor forces and the horizontal subgrade reaction force is for the condition where the soil at the bottom of the wall is not near a state of plastic equilibrium (i.e., failure), as discussed in Section 4.2.2 of FHWA-RD-97-130 and Section 5.8.2 of FHWA-SA-99-015. For a wide, infinitely long, excavation in a homogenous soft to medium clay of constant shear strength, the factor of safety is given by

$$FS = \frac{N_c}{\gamma \bullet \dfrac{H}{S_u}} = \frac{5.14}{N_s}$$

Table 3.5
Summary of Results for Eight-Tier, 50-ft Drilled-In Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Retaining Cohesive Soil–Stringent Displacement Control Design

Parameter		Value
Wall height		50 ft
Soldier beam spacing		6 ft
Soldier beam design moment		80 kip-ft
Soldier beam size		2 MC10×22
Soldier beam length		63 ft
Drill shaft diameter		26 in.
Toe reaction		9.97 kips
Top-tier anchor	H_1	6 ft, 3 in.
	Anchor inclination	20 deg
	Design load	66 kips
	Unbonded length	44.1 ft
	Bonded length	40 ft
	Total length	85 ft
	Tendon size	two 0.6-in. strands
Second-tier anchor	H_2	6 ft, 3 in.
	Anchor inclination	20 deg
	Design load	56.6 kips
	Unbonded length	39.2 ft
	Bonded length	40 ft
	Total length	80 ft
	Tendon size	two 0.6-in. strands
Third-tier anchor	H_3	6 ft, 3 in.
	Anchor inclination	20 deg
	Design load	56.6 kips
	Unbonded length	34.4 ft
	Bonded length	40 ft
	Total length	75 ft
	Tendon size	two 0.6-in. strands
Fourth-tier anchor	H_4	6 ft, 3 in.
	Anchor inclination	20 deg
	Design load	56.6 kips
	Unbonded length	29.5 ft
	Bonded length	40 ft
	Total length	70 ft
	Tendon size	two 0.6-in. strands
Fifth-tier anchor	H_5	6 ft, 3 in.
	Anchor inclination	20 deg
	Design load	56.6 kips
	Unbonded length	24.6 ft
	Bonded length	40 ft
	Total length	65 ft
	Tendon size	two 0.6-in. strands
Sixth-tier anchor	H_6	6 ft, 3 in.
	Anchor inclination	20 deg
	Design load	56.6 kips
	Unbonded length	19.7 ft
	Bonded length	40 ft
	Total length	60 ft
	Tendon size	two 0.6-in. strands
Lower-tier anchor	H_7	6 ft, 3 in.
	Anchor inclination	15 deg
	Design load	53.9 kips
	Unbonded length	15.1 ft
	Bonded length	40 ft
	Total length	56 ft
	Tendon size	two 0.6-in. strands

where

γ = total unit weight

N_s = stability number

Recall the stability number N_s has been used to identify excavation support systems with potential for movement and basal heave problems (Table 8.7 in Strom and Ebeling 2001; Table 12 in FHWA-SA-99-015). In this problem N_s is

$$N_s = \gamma \bullet \frac{H}{S_u} = 0.132 \bullet \frac{50}{2.4} = 2.75$$

Small values of N_s, with respect to a value of 5.14, indicate basal stability and small ground movements. The factor of safety against basal heave is

$$FS = \frac{5.14}{\mathrm{N_s}} = \frac{5.14}{2.75} = 1.87$$

Current practice according to FHWA-RD-97-130 is to use a minimum factor of safety of 1.5 against basal heave. A more detailed discussion is presented in Sections 54.3 and 37.3.1 of Terzaghi, Peck, and Mesri (1996). Cacoilo, Tamaro, and Edinger (1998) indicate a minimum safety factor of 1.5 is desirable to limit soil displacements. FHWA-SA-99-015 (page 107) notes that as the factor of safety decreases, loads on the lowest anchor increase. FHWA-SA-99-015 suggests a minimum factor of safety against basal heave of 2.5 for permanent facilities and 1.5 for support excavation facilities. Factors of safety below these target values indicate that more rigorous procedures such as limit equilibrium methods or Henkel's method should be used to evaluate design earth pressure loadings according to FHWA-SA-99-015 (page 107).

For weak soils (soft clays and loose sands) the failure surface will extend below the bottom of the cut, rather than through the bottom corner of the cut (FHWA-RD-98-065). For clay soils the deeper failure plane condition (i.e., below the bottom of the cut condition) can be evaluated by the Bishop method. The Bishop method is in reasonable agreement with the Henkel method (FHWA-SA-99-015). The Bishop method can be used in a GPSSP to determine the total load the tieback system must carry to meet the factor of safety requirements established for the project. The total load determined from a Bishop method internal stability limiting equilibrium analysis can be redistributed into an apparent pressure diagram. This apparent pressure diagram should be used as a basis for design if it provides a greater total load than that obtained from either a conventional apparent pressure diagram that assumes a "bottom corner of the cut" failure condition, or from an apparent pressure diagram constructed for the drained (long-term) condition. GPSSP analyses are described in FHWA-RD-98-065 and in Strom and Ebeling (2002b). GPSSP analyses should always be used to verify that the total load required to meet internal stability safety requirements is equal to or less than that used for the original design.

As the soil above the failure plane attempts to move out, shear resistance is mobilized in the soldier beams. Shear in the soldier beams provides additional resistance to soil movement. Predicting soldier beam shear requires the consideration of three possible failure modes: (1) shear in the soldier beam; (2) flow of the soil between the soldier beams; and (3) lateral capacity of the soldier beams. These three failure mechanisms are discussed in Section 4.3.3 of FHWA-RD-98-065. The resistance provided by the soldier beams for each of the three possible failure modes can be estimated and included in a GPSSP analysis, provided the GPSSP used is capable of modeling the soldier beams as reinforcement.

The calculated mass stability (i.e., external stability) slip circles for soft clays can be deep, and are generally located beyond or at the end of the tieback anchorage zone (Cacoilo, Tamaro, and Edinger 1998). Possibly, ground mass stability can be improved by increasing the depth of the soldier beam or by extending the length of the tiebacks, although significant improvement in the factor of safety can sometimes be difficult to obtain by these methods (Cacoilo, Tamaro, and Edinger 1998).

4 Simplified Design Procedures for 50-ft-High Vertical Sheet Piles with Wales and Post-Tensioned Tieback Anchored Wall System Retaining Cohesive Soil

The two example problems presented in this chapter deal with the application of the design procedures and guidelines for sheet-pile tieback wall systems given in Strom and Ebeling (2001), FHWA-RD-97-130, and FHWA-SA-99-015. Section 4.2.1.1 in FHWA-RD-97-130 discusses the applicability of the apparent pressure diagram-based approach to the design of tiebacks for ground anchor walls built from the top down using multiple rows of anchors for both soldier beam and lagging tieback wall systems as well as sheet-pile tieback wall systems. Section 5.4.1 in FHWA-SA-99-015 indicates that multi-anchored sheet pile walls (constructed by the top down method) like anchored soldier beam and lagging walls are to be designed to resist lateral loads resulting from apparent pressure envelopes.

A 50-ft wall height (horizontal retained soil surface) with homogeneous cohesive retained soil is considered. These design computations for the drilled-in soldier beam cohesive soil design example of Section 10.2 in FHWA-RD-97-130 have been adapted to this tieback sheet-pile wall design problem. A "safety with economy" design example is given first, followed by a "stringent displacement control" design example.

4.1 Soil Property Summary

This particular wall is founded in stiff clay. A stiff clay site was selected because soft to medium clay soils with stability numbers (γ H/S_u) greater than 5 are considered to be potentially dangerous and, as such, the use of a soldier beam and lagging system for support is questionable (see Table 12 of FHWA-SA-99-

015). It is likely that this limiting criterion would also be applicable to tieback sheet-pile walls for the same reason the criterion applies to soldier beam tieback walls. The soil properties selected are per the “Cohesive Soil Design Example” of FHWA-RD-97-130 (Step 2, page 204). The undrained shear strength (S_u) was given as 2,400 psf in the FHWA report for this homogeneous soil site. Using Figure 31 of FHWA-RD-97-130, the EPF for the undrained condition was estimated. For the 50-ft-high wall example calculation to be discussed in the following paragraphs, the EPF is equal to 20 psf, for S_u equal to 2,400 psf by this figure. This is for the short-term loading condition.

For clays, both the undrained (short-term) and drained (long-term) conditions must be evaluated. In the FHWA-RD-97-130 cohesive design example no long-term (drained) shear strength value was provided. FHWA-RD-97-130 estimated the drained shear strength for the long-term condition based on an empirical correlation. This same approach is used in the two design examples given in this chapter. This information is repeated in Appendix A of this report. The clay soil has a plasticity index of 19 and an overconsolidation ratio of 3, according to the FHWA problem statement (Step 2, page 204, FHWA-RD-97-130). It can be estimated—as shown in this report (Appendix A, Figure A.4, and also in the FHWA example)—that the drained friction angle for the long-term condition is equal to 36 degrees. (Note that no effective cohesion intercept is included in the Appendix A empirical correlation for both normally consolidated and overconsolidated cohesive soils by this correlation. For further explanation regarding this issue, the reader is referred to Appendix A.) As will be shown in the following calculations, the long-term condition governs the EPF value to be used in determining the design prestress anchor forces.

The soil properties used are in accordance with the cohesive soil, from examples given on in FHWA-RD-97-130 (page 204):

- Undrained shear strength S_u = 2,400 psf.
- Unit weight, γ = 132 pcf.
- Earth pressure factor for undrained (short-term) condition, EPF = 20 pcf.
- Friction angle for drained (long-term) condition ϕ = 36 deg.

4.2 “Safety with Economy” Design

For the Corps’ “safety with economy” design, a limiting equilibrium approach is used with a factor of safety of 1.3 applied to the shear strength of the soil. (The factor of safety for the limiting equilibrium analysis is increased to 1.5 for the stringent displacement control design.) The total earth pressure load (P_{tl}) is determined based on the limiting equilibrium analysis. Limiting equilibrium calculations for the safety with economy design are provided below.

$$\phi_{mob} = \tan^{-1}(\tan\phi / FS)$$

Accordingly,

$$\phi_{mob} = \tan^{-1}\left(\frac{\tan\phi}{1.3}\right)$$

$$= \tan^{-1}\left(\frac{\tan 36°}{1.3}\right) = 29.2°$$

$$K_A = \tan^2\left(45° - \frac{\phi_{mob}}{2}\right) = 0.344$$

$$P_{tl} = K_A \gamma \frac{H^2}{2} = 0.344 * 132 * \frac{50^2}{2} = 56760 \text{ lb/ft}$$

$$\text{Effective pressure factor, EPF} = \frac{P_{tl}}{H^2} = 22.7 \text{ lb/ft}^3$$

This calculation produces an EPF equal to 22.7 pcf for the long-term (drained) condition. Figure 31 in FHWA-RD-97-130 produces an EPF equal to 20 pcf for the short-term (undrained) condition. Use an EPF equal to 22.7 pcf in the construction of the apparent pressure diagram and in all subsequent computations involving the prestress design anchor forces. This design approach follows the steps taken in the FHWA-RD-97-130 cohesive soil design example of Section 10.2.1 (pages 202-213).

4.2.1 Anchor system

As noted in Section 1.5, a minimum of four rows of anchors is assumed. Further, the soil properties indicate stiff clay (see Section 3.2.1).

4.2.2 Anchor points

Using the empirical apparent earth pressure envelope (Figure 5.4, Strom and Ebeling 2001, and Figure 29, FHWA-RD-97-130), the vertical anchor intervals with four-tier anchoring for approximate balanced moments are determined.

$$\frac{1}{10} H^2{}_{(2,3,4,5)} = \frac{13}{54} H_1^2$$

$$H_{(2,3,4,5)} = \sqrt{\frac{130}{54}} H_1 = 1.55 H_1$$

i.e.,

$$H = H_1 + H_2 + H_3 + H_4 + H_5 = H_1 + 4(1.55 H_1)$$

$$50 = 7.2 H_1$$

$$\therefore H_1 = 6.94 \text{ ft}$$

$$H_{(2,3,4,5)} = 1.55 * 6.94 = 10.757 \text{ ft}$$

Try $H_2 = H_3 = H_4 + H_5 = 10$ ft, 6 in. and $H_1 = 8$ ft, 0 in.

Check cantilever and span deformations (see Equations 9.1 and 9.2, FHWA-RD-97-130).

These anchor spacings will be evaluated using Equations 9.1 and 9.2 (FHWA-RD-97-130) to determine if the associated cantilever and interior spans can also be used to meet stringent displacement control performance requirements.

Approximate cantilever deformation y_c allowing 1.5 ft overexcavation for placement of top anchor, $h_1 = 8 + 1.5 = 9.5$ ft and with $E_s = 2850$ psi for stiff clay, and $K_o = 0.5$,

$$y_c = \frac{4K_0 \gamma h_1^2}{E_s} = \frac{4 * 0.5 * 132 * 9.5^2}{2850 * 12} = 0.697 \text{ in.} > 0.5 \text{ in.}$$

The soil modulus (E_s) was obtained from Table 20 of FHWA-RD-97-130.

Approximate span bulging deformation y_b with h = 10.5 ft and wall height = 50 ft

$$y_b = \frac{0.8K_0 \gamma h L}{E_s} = \frac{0.8 * 0.5 * 132 * 10.5 * 50}{2850 * 12} = 0.81 \text{ in.} > 0.5 \text{ in.}$$

Deformations are larger than the 1/2-in. maximum for stringent displacement control, but are not excessive for safety with economy design requirements.

4.2.3 Apparent earth pressure

The effective earth pressure (p_e) based on Figure 5.4 (Strom and Ebeling 2001) is

$$p_2 = \frac{P_{tl}}{H - \frac{H_1}{3} - \frac{H_5}{3}}$$

$$= \frac{56760}{50 - \frac{8}{3} - \frac{10.5}{3}} = 1295 \text{ psf}$$

4.2.4 Horizontal components of anchor loads

From Figure 5.4b (Strom and Ebeling 2001), the horizontal component of each anchor load T_i is determined. Anchor loads are expressed in pounds per foot run of wall.

$$T_1 = \left(\frac{2}{3}H_1 + \frac{1}{2}H_2\right)p = \left(\frac{2}{3}*8 + \frac{1}{2}*10.5\right)*1295 = 13705.4\ \text{lb/ft}$$

$$T_{(2,3)} = \left(\frac{H_2}{2} + \frac{H_3}{2}\right)p = \left(\frac{10.5}{2} + \frac{10.5}{2}\right)*1295 = 13597.5\ \text{lb/ft}$$

$$T_4 = \left(\frac{H_4}{2} + \frac{23}{48}H_5\right)p = \left(\frac{10.5}{2} + \frac{23}{48}*10.5\right)*1295 = 13314.2\ \text{lb/ft}$$

$$T_{max} = 13705.4\ \text{lb/ft}$$

4.2.5 Anchor loads (TF)

For constructibility, an anchor inclination of 10 degrees to the horizontal will be used, and the total anchor force (TF) per foot of wall determined. Assumed anchor spacing = 8.8 ft.

Top tier:

$$TF_1 = \frac{T_1}{\cos 10°} = \frac{13705.4}{\cos 10°} = 13916.8\ \text{lb/ft}$$

(Design anchor force = 13.92 kips/ft × 8.8 ft = 122.5 kips)

Tiers 2, 3:

$$TF_{(2,3)} = \frac{T_{(2,3)}}{\cos 10°} = \frac{13597.5}{\cos 10°} = 13807.3\ \ \text{lb/ft}$$

(Design anchor force = 13.81 kips/ft × 8.8 ft = 121.5 kips)

Lower tier:

$$TF_4 = \frac{T_1}{\cos 10°} = \frac{13705.4}{\cos 10°} = 13916.8\ \text{lb/ft}$$

(Design anchor force = 13.92 kips/ft × 8.8 ft = 122.5 kips)

Anchor loads are approximately balanced.

Use TF_{max} = 13,916.8 lb/ft

4.2.6 Subgrade reaction using tributary method

From Figure 5.4b (Strom and Ebeling 2001), the subgrade reaction (R) is determined. The subgrade reaction is expressed in pounds per foot of wall.

$$R = \left(\frac{3}{16}H_5\right)p = \left(\frac{3}{16}*10.5\right)*1295 = 2549.5 \text{ lb/ft}$$

4.2.7 Bending moments

Using the information contained in Figure 5.4b (Strom and Ebeling 2001), the cantilever moment (M_1) and the maximum interior span moments (MM_1) can be determined. Moments are per foot of wall.

Negative moment at point of top anchor is

$$M_1 = \frac{13}{54}H_1^2 p = \frac{13}{54}*8.0^2*1295 = 19952.6 \text{ ft} - \text{lb/ft}$$

Maximum moment below top-tier anchor,

$$MM_{(1,2,3)} = \frac{1}{10}H^2{}_{\max}p \qquad (H_{\max} \text{ is the larger of } H_2, H_3, H_4)$$

$$= \frac{1}{10}*10.5^2*1295 = 14277.4 \text{ ft} - \text{lb/ft}$$

NOTE: moments are not well balanced but, noting that anchor loads are well balanced, vertical anchor spacing need not be revised.

USE design moment M = 19952.6 ft - lb/ft.

4.2.8 Design of vertical sheet-pile system components

4.2.8.1 Select economical AZ-type sheet pile. In accordance with Corps criteria (HQUSACE 1994), the allowable stresses for the sheet piling and wales shall be as follows:

Bending (i.e., combined bending and axial load) $f_b = 0.5 f_y$

Shear $f_v = 0.33 f_y$

Allowable stresses are based on 5/6 of the AISC-ASD recommended values (AISC 1989) and reflect the Corps' design requirements for steel structures.

Try AZ 13 ARBED Hot-Rolled Sheet Piles with

Section modulus about bending axis, S_x = 24.2 in.^{3}/ft

Width per sheet, w = 26.38 in.

Moment on sheet pile = 19,952.6 lb-ft/ft

Allowable bending stress for Grade 50 steel f_b = 25 ksi

Allowable shear stress for Grade 50 steel f_v = 16.5 ksi

Required section modulus = 19.9526*12/25 = 9.58 in.^{3} < 24.2 in.^{3} OK

Check shear capacity:

Maximum shear force, $V_{max} = T_{max}$ = 13.7 kips/ft

Required area, $A = \frac{13.7}{16.5} = 0.83 \text{ in.}^2$ per ft run

Shear area provided by an AZ 13 (Equation 6-5 in EM 1110-2-2504 (HQUSACE 1994))

$$= \frac{t_w \bullet h}{w} = \frac{0.375 \text{ in.} \bullet 11.93 \text{ in.}}{26.38 \text{ in.} \bullet \frac{\text{ft}}{12 \text{ in.}}}$$

$$= 2.04 \text{ in.}^2 \text{ per ft run} > 0.83 \text{ in.}^2 \text{ per ft run} \qquad \text{OK}$$

where

t_w = thickness of the web portion of the Z = 0.375 in.

h = height of the Z = 11.93 in.

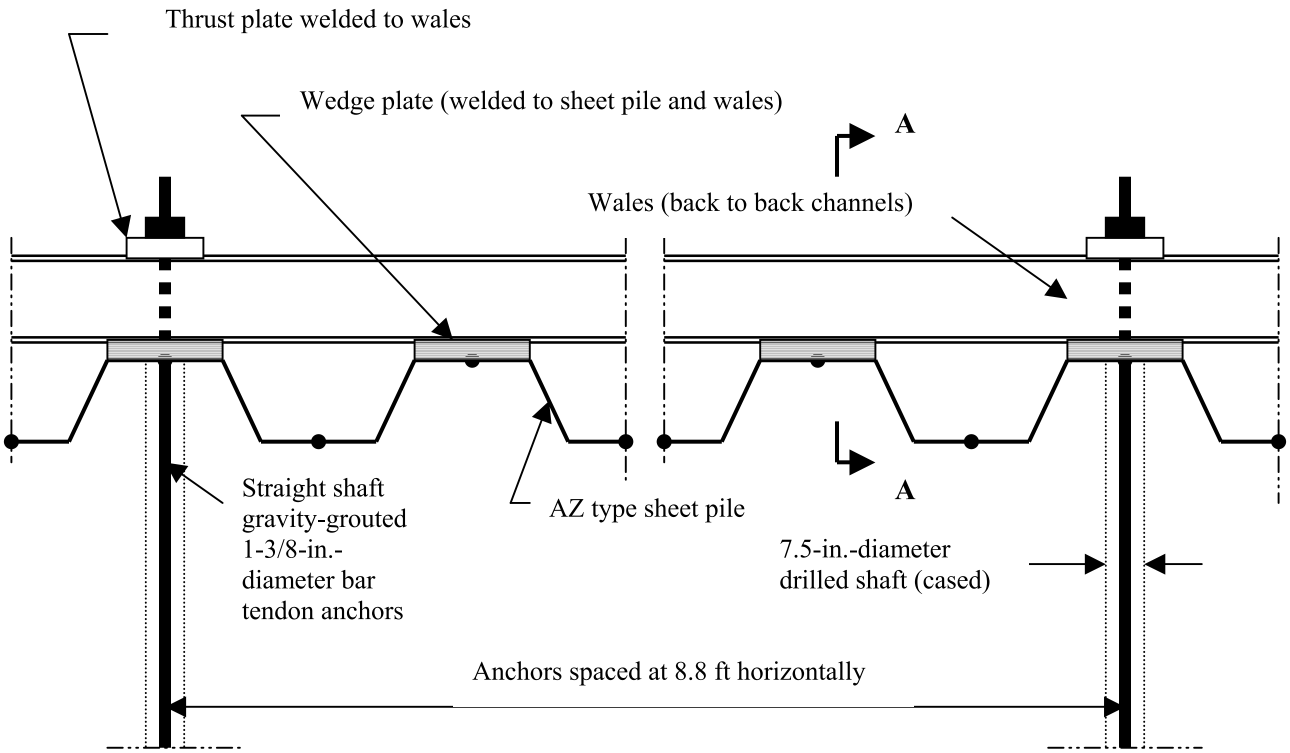
<u>Use</u> AZ 13 Grade 50 sheetpiling.

4.2.8.2 Select economical bar tendon. A 150 grade prestressing steel bar will be selected from Table 8.4 of Strom and Ebeling (2001) to meet "safety with economy" design requirements. Anchors will be spaced to occur at the center of every fourth pair of z-section sheet piling (i.e., anchor spacing = 4 (26.38) = 105.52 in. = 8.8 ft). It is assumed that the final anchor prestress force (after losses) will equal 0.6 f_{pu} A_{ps}, where:

f_{pu} = anchor ultimate tensile strength = 150 ksi

A_{ps} = Cross-sectional area of bar tendon (in.^{2})

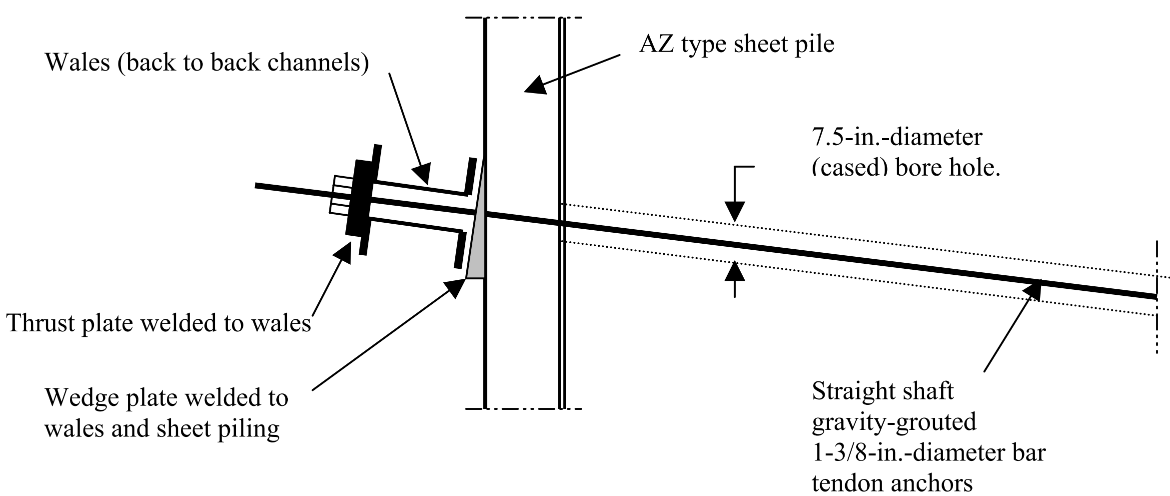
Bar tendons, rather than wire-strand tendons, are used to facilitate construction of the sheet pile-wale-anchor system. Details of this system are illustrated in Figure 4.1.



Construction sequencing

- Drive sheet piling driven to required depth.
- First stage (cantilever) excavation performed.
- Drill 7.5-in.-diameter (cased) bore hole.
- Place bar tendons, grout anchor zone and unbonded zone.
- Place walers, install anchor plates, stress and lock-off tendons.
- Repeat process for each excavation stage.

a. Horizontal section



b. Vertical section through sheet piling (Section A-A)

Figure 4.1. Sheet pile-wale-anchor system details

As indicated in Figure 4.1 a 7.5-in.-diameter cased borehole will be used to place the bar tendons. The casing will be pulled as grouting takes place. Bar tendons are over 60 ft long, so a coupler will be needed.

Total anchor load required (TL) = 13,916.8 (8.8) = 122,467 lb = 122.5 kips

From Table 8.4 of Strom and Ebeling (2001) a 1-3/8-in.-diameter, 150 grade bar tendon at 0.6 f_{pu} A_{ps}, can provide a final prestressing force up to 142.2 kips > 122.5 kips OK.

4.2.8.3 Select wales. The wales are positioned on the outside of the sheet pile as shown in Figure 4.1. The design moment for continuous wales can be approximated using Equation 6-14 of EM 1110-2-2504 (HQUSACE 1994).

$$M_{Max} = \frac{T_{ah} S^2}{10}$$

where

T_{ah} = anchor force per foot of wall = 122.5 ÷ 8.8 = 13.92 kips per foot of wall

S = distance between adjacent anchors = 8.8 ft

$$M_{Max} = \frac{T_{ah} S^2}{10} = \frac{13.92(8.8)^2}{10} = 107.8 \text{ ft-kips}$$

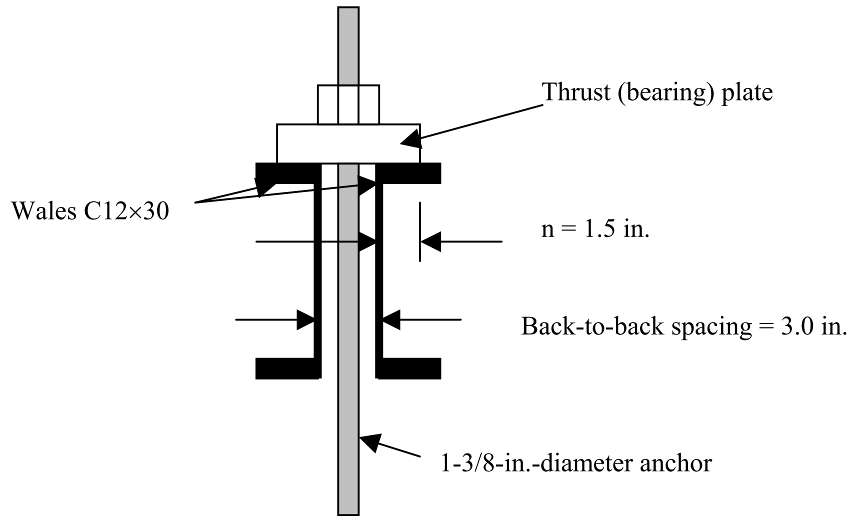
The allowable stress design provisions of AISC (1989) will be used in accordance with Corps criteria as specified in EM 1110-2-2504. As such allowable stresses, or allowable loads, will be 5/6 of the appropriate AISC-ASD requirement (AISC 1989).

Using 50 grade steel, the required section modulus (S_x) assuming an allowable bending stress of 5/6 × 0.60 F_y, or 0.5 (50 ksi) is:

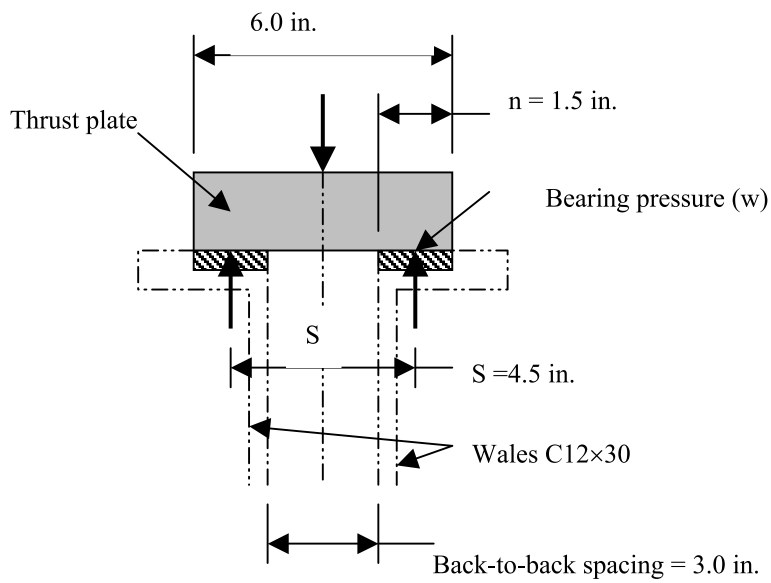
$$S_x = \frac{107.8(12)}{25} = 51.7 \text{ in.}^3$$

Two C12×30 channels back-to-back have a section modulus of 2(27.0) = 54.0 in.^{3} > 51.7 in.^{3} OK. Space channels at 3.0 in. back-to-back (See Figure 4.1).

4.2.8.4 Select thrust (bearing) plate. Try a 6-in.-wide × 6-in.-long × 2.5-in.-deep thrust plate. See Figure 4.2 for details.



a. Thrust plate detail



b. Thrust plate forces

Figure 4.2. Thrust plate

The bearing pressure (w) exerted by the thrust plate on each wale is:

$$w = \frac{TL}{2Bn} = \frac{125.5}{2(6)1.5} = 6.972 \text{ kips per in.}^2$$

where

B = bearing plate width = 6 in.

n = bearing contact width on each wale = 1.5 in.

TL = total anchor load = 125.5 kips

The maximum moment on the plate (M_{PL}) is:

$$M_{PL} = \frac{TL(S)}{4} = \frac{125.5(4.5)}{4} = 141.19 \text{ in.-kips (See Figure 4.2b)}$$

Using 50 grade steel, the required section modulus (S_x) assuming an allowable bending stress of 5/6 × 0.60 F_y, or 0.5 (50 ksi) is:

$$S_x = \frac{141.19}{25} = 5.65 \text{ in.}^3$$

Section modulus provided (S_{PL}) is:

$$S_{PL} = \frac{Bd^2}{6} = \frac{6(2.5)^2}{6} = 6.25 \text{ in.}^3 > 5.65 \text{ in.}^3 \text{ OK}$$

Use 6-in.-wide × 6-in.-long × 2.5-in.-deep thrust plate.

Checking shear in the thrust plate:

$$f_v = \frac{TL}{2Bd} = \frac{125.5}{(2)(6)(2.5)} = 4.2 \text{ ksi} < \frac{5}{6}(0.40\ Fy) = 16.67 \text{ ksi} \quad \text{OK}$$

4.2.8.5 Check web yielding. In accordance with AISC-ASD Equation K-2 (AISC 1989) the maximum interior load reaction for web yielding (R) is:

$R = 5/6(0.66\ F_y)\ t_w\ (N + 5k)$

where

N = bearing length = 6 in.

k = distance from top flange surface to web toe of fillet

= 1.125 in. for C12×30 channel

t_w = web thickness = 0.51 in. for C12×30 channel

R = 0.55 (50) (0.51) [6 + 5(1.125)] = 163.0 kips > 62.75 kips OK

4.2.8.6 Check web crippling. In accordance with AISC-ASD Equation K1-4 (AISC 1989) the maximum concentrated load (L_{CR}) for web crippling is:

$$L_{CR} = 67.5t_w^2\left[1+3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_y\left(\frac{t_f}{t_w}\right)}$$

where

d = overall depth of member = 12.00 in. for C12×30

t_f = flange thickness = 0.50 in. for C12×30 channel

Maximum concentrated load (L_{CR}) is proportional to stress. Since a 5/6 reduction is being used to obtain allowable stress, 5/6 of maximum web crippling load (L_{CR}) is used.

$$L_{CR} = \frac{5}{6}\left\{67.5(0.51)^2\left[1+3\left(\frac{6}{12}\right)(1.02)^{1.5}\right]\sqrt{50(0.98)}\right\}$$
$$= 261.2 \text{ kips} > 62.75 \text{ kips OK}$$

4.2.8.7 Check web compression buckling. In accordance with AISC-ASD Equation K1-8 (AISC 1989) it can be determined whether or not web stiffeners are required to prevent compression buckling of the C12×30 channel web.

$$\frac{4100(t_w)^3\sqrt{F_y}}{P_{bf}} = \frac{4100(.510)^3\sqrt{50}}{\frac{5}{3}(61.25)} = 37.7 \text{ in.} < d_c = 9.75 \text{ in.}$$

OK - stiffeners not required.

where

P_{bf} = the computed force delivered by the flange (= 122.5/2) multiplied by 5/3.

d_c = d – 2k = 9.75 in. for C12×30 channel.

4.2.8.8 Check web sidesway buckling. The outside flanges of the C12×30 channels are to be welded to the thrust plate and the inside flanges are welded to a wedge plate that in turn is welded to the sheet piling (see Figure 4.1). With this construction detail, in conjunction with the use of a prestressed bar tendon, the C12×30 channels are likely to be braced against sidesway at the point of load application by the bar tendon (prestressed in tension), and sidesway buckling is not likely to occur. However, should sidesway be of concern to the designer, Equation K1-6, given in AISC-ASD Chapter K, section K1, subsection 5 (AISC 1989) can be used to determine whether or not sidesway buckling is an issue for the loaded flange (of the C-channel) restrained against rotation. (Equation K1-7 is for a loaded flange not restrained against rotation.)

4.2.9 Anchor lengths

4.2.9.1 Unbonded anchor length, L_i. Assume 10-degree inclination for all anchors. The unbonded length must be sufficient such that anchor bond zone is beyond the short-term (undrained) and long-term (drained) potential failure surfaces and satisfy the Figure 8.5, Strom and Ebeling (2001), length criteria. With the short-term shear strength characterized in terms of S_u equal to 2,400 psf (with $\phi = 0$ degree), and with the long-term shear strength characterized in terms of ϕ' equal to 36 degrees, the short-term loading condition will require greater unbonded anchor lengths. Thus, the potential failure plane will be based on the undrained shear strength with $\phi = 0$ degree (as is also done in the FHWA-RD-97-130 cohesive design example; refer to Figure 107).

$$\frac{\text{unbonded length } L}{(45° - \phi/2)} = \frac{\text{height of anchor point}}{\alpha} \qquad (\text{see Figure 4.3})$$

$$45° - \frac{\phi}{2} = 45° - \frac{0°}{2} = 45°$$

$$\alpha = 180° - 45° - 80° = 55°$$

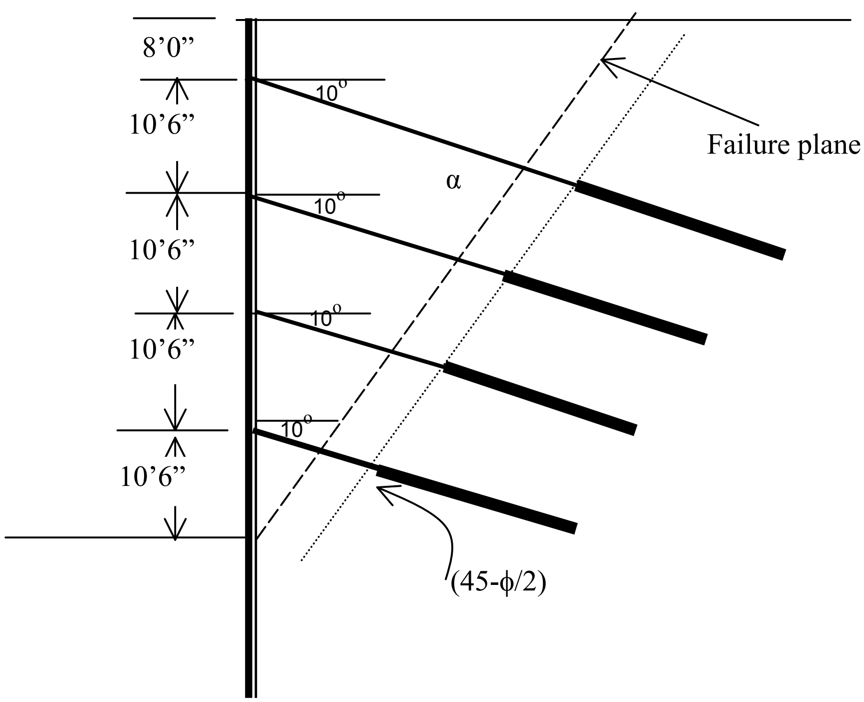


Figure 4.3. Four-tier anchors and placements

Top-tier anchor:

$$\frac{L}{\sin 45} = \frac{42.00}{\sin 55}$$

$$L = \frac{42.00 * \sin 45}{\sin 55} = 36.26 \text{ ft}$$

Allowing 5 ft or 0.2H clearance beyond shear plane (see Figure 8.5, Strom and Ebeling 2001).

$$\begin{aligned} L_1 &= 36.26 + (0.2H \text{ or 5 ft, whichever is greater}) \\ &= 36.26 + 10 = 46.26 \text{ ft} > 10 \text{ ft minimum required for bar anchor} \quad \text{OK} \\ &\quad \text{(Minimum required for strand anchor is 15 ft)} \end{aligned}$$

Similarly, second-tier anchor:

$$\begin{aligned} L_2 &= \frac{31.5}{42} * L + 0.2H \\ &= 37.19 \text{ ft} > 15 \text{ ft} \quad \text{OK} \end{aligned}$$

Third-tier anchor:

$$\begin{aligned} L_3 &= \frac{21.0}{42} * L + 0.2H \\ &= 28.13 \text{ ft} > 15 \text{ ft} \quad \text{OK} \end{aligned}$$

Lower-tier anchor:

$$\begin{aligned} L_4 &= \frac{10.5}{42.0} * L + 0.2H \\ &= 19.06 \text{ ft} > 15 \text{ ft} \quad \text{OK} \end{aligned}$$

The unbonded length determined above should be verified using the internal stability analysis procedures described for both undrained (short-term) and drained (long-term) conditions in Strom and Ebeling (2002b). The verification process uses limiting equilibrium procedures, which can be performed by simple hand calculations or general-purpose ground slope stability (GPSS) procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to meet internal stability performance requirements (i.e., factor of safety of 1.3 for a "safety with economy" design).

4.2.9.2 Bonded length of anchors, L_b. The usual practice is for the wall designer to specify the anchor capacity and any right-of-way and easement constraints required of the anchorage system. It is up to the tieback anchor contractor, usually a specialty subcontractor, to propose the type of anchorage system to be used to meet the wall design requirements. A preliminary estimate of the bond length (L_b) required to develop the ground anchors is provided below.

The horizontal anchor force T_1 corresponds to maximum horizontal anchor force T_{max} (Section 4.2.4). Because the horizontal anchor forces T_2 , T_3 and T_4 are within 3 percent of this T_{max} value, the bond length computations will be made using the tendon force value of T_{max}. The computed bond length will be slightly conservative for anchor tendons 2, 3, and 4. With 8.8-ft horizontal spacing between anchors the maximum anchor (tendon) force, AL, is

$$AL = 13{,}916.4 * 8.8 = 122{,}468 \text{ lb} = 122.5 \text{ kips}$$

An empirical method (Equation 1.2) was attempted to estimate the bond length of large-diameter straight-shaft gravity-grouted anchors for preliminary design purposes. Recall that it is up to the tieback anchor contractor, usually a specialty subcontractor, to propose the type of anchorage system to be used to meet the wall design requirements.

The design tendon force for the anchor bond zone is computed equal to 122.5 kips. Applying a factor of safety equal to 2.0 to this design force results in an ultimate anchor force equal to 245 kips.

No site-specific anchor load test data are available for use in this design. Consequently, preliminary design computations are made using traditional (non-site specific) assumptions for the range in value of the adhesion factor when computing an average ultimate soil-to-grout bond stress (Equation 1.3) and subsequently, the ultimate capacity of a large-diameter straight-shaft gravity-grouted anchor of 40-ft length (the maximum possible length without requiring specialized methods). The ultimate anchor capacity of a large-diameter straight-shaft gravity-grouted anchor was computed using Equation 1.2. These computations (not shown) indicate that a more robust anchorage system will be required to achieve an ultimate anchorage capacity of 245 kips.

Review of information contained in Section 1.5.4 indicates that a post-grouted (regroutable) ground anchor is a possible solution. (The anchor capacities cited for the case histories given and for the soil conditions cited in this section makes this type of anchorage system a viable candidate.) For an ultimate anchor force equal to 245 kips and assuming a 40-ft bond length, the ultimate capacity of rate of load transfer corresponds to 6.13 kips per lineal ft. A preliminary bonded length L_b of 40 ft will be assumed for anchorage layout purposes. Recognize that the final bond length, anchor capacity, etc., will be confirmed by proof-testing and performance testing onsite (Strom and Ebeling 2002b).

4.2.9.3 Total anchor lengths ($Lt_i = L_i + L_b$).

Top-tier anchor:

$$Lt_1 = 46.26 + 40 = 86.26 \text{ ft} \approx 87 \text{ ft}$$

Second-tier anchor:

$$Lt_2 = 37.19 + 40 = 77.19 \text{ ft} \approx 78 \text{ ft}$$

Third-tier anchor:

$$Lt_3 = 28.13 + 40 = 68.13 \text{ ft} \approx 69 \text{ ft}$$

Lower-tier anchor:

$$Lt_4 = 19.06 + 40 = 59.06 \text{ ft} \approx 60 \text{ ft}$$

The total anchor length (bonded + unbonded length) determined above should be verified using the external stability analysis procedures described in Strom and Ebeling (2002b) for both short-term (undrained) and long-term (drained) conditions. This verification process also uses limiting equilibrium procedures, which can be performed by simple hand calculations or GPSS procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to prevent ground mass stability failure (i.e., meet external stability performance requirements with factor of safety of 1.3 for a "safety with economy" design).

4.2.10 Determine required depth of sheet pile penetration, D

Passive resistance mobilized in front of the toe must be adequate to resist the reaction with a factor of safety of 1.5 (Section 8.7.1 of Strom and Ebeling 2001; Section 6.2 of FHWA-RD-97-130).

4.2.10.1 Short-term undrained condition. For a continuous wall, the passive earth pressure force, P_p, per ft run of wall is

$$P_p = (2 \bullet S_u + \gamma \bullet D + 2 \bullet S_u) \bullet \frac{D}{2} = \left(2 \bullet S_u \bullet D + \frac{\gamma \bullet D^2}{2} \right)$$

According to FHWA-RD-97-130, page 104, the Rankine active pressures must be applied to the other side (i.e., the retained side) of the wall when computing the capacity of the toe. Thus, the usable net resistance is given by

$$P_p - P_a = R \bullet FS$$

where the factor of safety, *FS*, is set equal to 1.5 (Section 8.7.1 of Strom and Ebeling 2001; Section 6.2 in FHWA-RD-97-130), R is equal to 2.4595 kips/ft (Section 4.2.6), and the active force, P_a, per ft run of wall is

$$P_a = (\gamma \bullet H - 2 \bullet S_u + \gamma \bullet (H + D) - 2 \bullet S_u) \bullet \frac{D}{2}$$

$$= \left[(\gamma \bullet H - 2 \bullet S_u) \bullet D + \frac{\gamma \bullet D^2}{2} \right]$$

The net usable resistance to (*R*)(*FS*) becomes

$$P_p - P_a = 4 \bullet S_u \bullet D - \gamma \bullet H \bullet D$$

Solve for D by setting the net usable passive resistance equal to the factored reaction force (*R*)(*FS*).

$$4 \bullet S_u \bullet D - \gamma \bullet H \bullet D = R \bullet FS$$

$$4 \bullet 2.4 \bullet D - 0.132 \bullet 50 \bullet D = 2.4595 \bullet 1.5$$

$$9.6 \bullet D - 6.6 \bullet D = 2.4595 \bullet 1.5$$

$$3 \bullet D = 3.6893$$

$$D = 1.23 \text{ ft}$$

4.2.10.2 Long-term drained condition. For a continuous wall, the passive earth pressure force, P_p, per ft run of wall is

$$P_p = K_p \bullet \gamma \bullet \frac{D^2}{2}$$

where conservatively assuming zero soil-to-steel interface friction, K_p by the Rankine relationship is

$$K_p = \tan^2\left(45° + \frac{\phi'}{2}\right) = \tan^2\left(45° + \frac{36°}{2}\right) = \tan^2(63°) = 3.85$$

According to FHWA-RD-97-130, page 104, the Rankine active pressures must be applied to the other side (i.e., the retained side) of the wall when computing the capacity of the toe. Thus, the usable net resistance is given by

$$P_p - P_a = R \bullet FS$$

where the factor of safety, FS, is set equal to 1.5 (Section 8.7.1 of Strom and Ebeling 2001; Section 6.2 in FHWA-RD-97-130), R is equal to 2.4595 kips/ft (Section 4.2.6), and the active force, P_a, per ft run of wall is

$$P_a = K_a \bullet [\gamma \bullet H + \gamma \bullet (H + D)] \bullet \frac{D}{2} = K_a \bullet \left[(\gamma \bullet H) \bullet D + \frac{\gamma \bullet D^2}{2} \right]$$

Assuming zero soil-to-steel interface friction, K_a (by the Rankine relationship), is

$$K_a = \tan^2\left(45° - \frac{\phi'}{2}\right) = \tan^2\left(45° - \frac{36°}{2}\right) = \tan^2(27°) = 0.26$$

The net usable resistance to (R)(FS) becomes

$$P_p - P_a = K_p \bullet \gamma \bullet \frac{D^2}{2} - K_a \bullet \left[(\gamma \bullet H) \bullet D + \frac{\gamma \bullet D^2}{2} \right]$$

Solve for D by setting the net usable passive resistance equal to the factored reaction force (R)(FS):

$$K_p \bullet \gamma \bullet \frac{D^2}{2} - K_a \bullet \left[(\gamma \bullet H) \bullet D + \frac{\gamma \bullet D^2}{2} \right] = R \bullet FS$$

$$(K_p - K_a) \bullet \gamma \bullet \frac{D^2}{2} - K_a \bullet (\gamma \bullet H) \bullet D = R \bullet FS$$

$$(3.85 - 0.26) \bullet 0.132 \bullet \frac{D^2}{2} - 0.26 \bullet (0.132 \bullet 50) \bullet D = 2.4595 \bullet 1.5$$

$$0.2369 \bullet D^2 - 1.716 \bullet D - 3.6893 = 0$$

The formula for the solution of a quadratic equation for D is

$$D = \frac{-(-1.716) \pm \sqrt{(1.716)^2 - 4 \bullet (0.2369) \bullet (-3.6893)}}{2 \bullet (0.2369)}$$

$$= \frac{1.716 \pm \sqrt{2.9447 + 3.496}}{0.4738} = \frac{1.716 \pm 2.5378}{0.4738} = 8.98 \text{ ft}$$

Use D = 9 ft penetration.

4.2.10.3 Sheet-pile toe embedment. Sheet-pile toe embedment requirements for both vertical and horizontal loads must be determined. With respect to the vertical component of prestress anchor load:

$$\sum V = (V_1 + V_2 + V_3 + V_4)$$

$$\sum V = (T_1 + T_2 + T_3 + T_4) \bullet \tan(10°) \bullet 8.8$$

$$\sum V = (13705.4 + 2 \bullet (13597.5) + 13314.2) \bullet \tan(10°) \bullet 8.8$$

$$\sum V = 84{,}125.5 \text{ lb} = 84.1 \text{ kips}$$

The anchors are spaced at 8.8-ft intervals.

The following computations are made to determine total force that the sheet-pile foundation must resist. A 9-ft depth of penetration is assumed in these computations for a 50-ft exposed wall height.

The total sheet-pile and wale weight assuming a 9-ft toe length is equal to the vertical component of anchor force plus the weight of the sheet-pile plus the weight of eight MC 12x30 channels used to form the four wales. The axial load transfer from the drilled shaft above the bottom of the wall to the retained soil, which acts upward on the sheet pile, is also included in the computations. The magnitude of each of these forces are summarized in the following steps:

a. Vertical component of anchor force = 84.1 kips, with an 8.8-ft anchor spacing. The vertical component of anchor force per ft run of wall = 9.56 kips per ft run of wall.

b. Computation of the axial load transfer from the sheet pile above the bottom of the wall to the retained soil:

(1) American Society of Civil Engineers (ASCE) Geotechnical Special Publication No. 74 by a Committee on Earth Retaining Structures states on page 108 (ASCE 1997) that since nonhorizontal tiebacks exert a downward (anchor) force on the wall (through the wales), that tiebacks with modest inclinations are usually preferable to steep ones. They also note that when the tieback wall settles, less horizontal movement occurs with flatter tiebacks. Lastly, vertical effects are minimized if the sheet-pile wall is adequate to transmit the vertical loads to soil beneath the excavation, or <u>if shear between the back of the sheeting and retained soil is adequate to provided the required vertical reaction</u>.

(2) Axial load and ground movements are interrelated. The magnitude of the axial load depends upon the vertical components of the ground anchor loads, the strength of the supported ground, vertical and lateral movements of the wall, the relative movements of the ground with respect to the wall, and the axial load carrying capacity of the toe. FHWA-RD-98-066 (page 66) discusses results taken from walls in dense sands and stiff to hard clays in which the axial load measured in the soldier beam toes was <u>less</u> than the vertical components of the ground anchors. This favorable axial load transfer

from the soldier beam to the retained soil is idealized in Figure 41(b) of FHWA-RD-97-130. Axial load transferred to the ground above the bottom of the excavations in stiff to hard clays was equal to A_s times ($0.25S_u$) according to FHWA-RD-97-130 (page 88). A_s was the surface area of the soldier beam in contact with the ground above the bottom of the excavation, and $0.25S_u$ was the back-calculated adhesion. At the stiff cohesive sites, the load transferred from the soldier beam to the ground above the bottom of the excavation appears to be valid for the long-term condition. Using an adhesion equal to 25 percent of the undrained shear strength gives a lower load transfer rate than a rate based on drained shear strengths. In adapting this to sheet-pile and wale retaining wall system, the subsequent calculations are made on a per ft run of wall basis. This computation assumes composite wall action for the sheet-pile and wale wall system.

(3) To take advantage of this axial load transfer from the sheet pile to the retained stiff to hard (cohesive) soil, Table 11 in FHWA-RD-98-066 and Table 11 in FHWA-RD-97-130 stipulate that

$$S_u > \frac{\gamma \bullet H}{4} - 5.714 \bullet H$$

which for this problem becomes

$$2{,}400 \text{ psf} > \frac{132 \text{ pcf} \bullet 50 \text{ ft}}{4} - 5.714 \bullet 50 \text{ ft}$$

$$2{,}400 > 1{,}650 - 285.7$$

$$2{,}400 > 1{,}364.3 \qquad OK$$

(4) So the following set of computations assumes that the axial load is transferred to the ground above the bottom of the excavation in this stiff clay site. Page 209 in FHWA-RD-97-130 gives this transfer force as

$$\text{Axial load transfer} = \alpha \bullet S_{.u} \bullet A_s \bullet \left(H - H_1\right)$$

where H is the height of the wall (= 50 ft) and H_1 is the depth to the first row of anchors (8 ft in Section 4.2.2). A_s is approximated as equal to 1 ft for a continuous, in-plan, sheet-pile system (rather than A_s equal to half the circumference of the drilled-in soldier beam shaft as used in the Section 3.2.11 computations).

$$\text{Axial load transfer} = 0.25 \bullet 2400 \text{ psf} \bullet 1 \text{ ft} \bullet \left(50 \text{ ft} - 8 \text{ ft}\right)$$

$$= 600 \text{ psf} \bullet 1 \text{ ft} \bullet 42 \text{ ft} = 25{,}200 \text{ lb} = 25.2 \text{ kips per ft run of wall}$$

(5) Note that this 25.2-kip force acts to <u>reduce</u> the axial load acting on the sheet-pile foundation. The magnitude of this upward-acting force

(from the perspective of the sheet pile) is significant. Great care must be exercised by the designers when taking advantage of this load transfer mechanism. It is assumed in this wall design that the soldier beam wall settles relative to the ground. Before applying this force in a design, designers should review the discussion and guidance given on pages 87-90 of FHWA-RD-97-130 and pages 66-69 in FHWA-RD-98-066. The instrumented wall case histories are discussed in FHWA-RD-98-066.

(6) It is important to recognize also that when these computations are made on a per foot run of wall basis, it is assumed that the entire sheet-pile wall (consisting of four AZ 13 sheets between each column of tieback anchors and the rows of wales) acts as a composite wall system. For this to occur, all of the sheeting between tieback anchors must contribute to the (upward-acting) vertical shear force provided by the retained soil as well as resist the vertical component of the anchor forces. One means of accomplishing this (vertical shear transfer between sheets) is the use of welded wedge plate connections between each sheet pile and each wale (see Figure 4.1a). A vertical shear transfer capacity calculation should be made by the wall designers to verify this composite wall assumption (calculations are not included in this example).

c. Weight of AZ 13 sheet-pile per ft run of wall = 59 ft * 0.02192 = 1.29 kips per ft run of wall.

d. Weight of 8 MC 12×30 channels for the four rows of wales = 8*0.03 = 0.24 kips per ft run of wall.

e. Computation of the applied total axial load per ft run of wall:

$$Q_{applied} = \sum V \text{ - Axial load transfer + Weight of sheet - piles}$$
$$\text{+ Weight of channels}$$

$$Q_{applied} = 9.56 \text{ kips} - 25.2 \text{ kips} + 1.29 \text{ kips} + 0.24 \text{ kips}$$
$$= -14.11 \text{ kips per ft run of wall}$$

Thus, for the continuous sheet-pile and wale retaining wall system with a 9-ft depth of penetration, there is no net downward applied axial load (as calculated on a per ft run of wall basis) due to the load transfer to the backfill.

4.2.11 Basal stability

The apparent pressure diagram used to compute the horizontal components of the anchor forces and the horizontal subgrade reaction force is for the condition where the soil at the bottom of the wall is not near a state of plastic equilibrium (i.e., failure), as discussed in Section 4.2.2 of FHWA-RD-97-130 and

Section 5.8.2 of FHWA-SA-99-015. For a wide, infinitely long, excavation in a homogenous soft to medium clay of constant shear strength, the factor of safety is given by

$$FS = \frac{N_c}{\gamma \bullet \dfrac{H}{S_u}} = \frac{5.14}{N_s}$$

where

γ = total unit weight

N_s = stability number

Recall the stability number N_s has been used to identify excavation support systems with potential for movement and basal heave problems (Table 8.7 in Strom and Ebeling 2001; Table 12 in FHWA-SA-99-015). In this problem N_s is

$$N_s = \gamma \bullet \frac{H}{S_u} = 0.132 \bullet \frac{50}{2.4} = 2.75$$

Small values of N_s, with respect to a value of 5.14, indicate basal stability and small ground movements. The factor of safety against basal heave is

$$FS = \frac{5.14}{N_s} = \frac{5.14}{2.75} = 1.87$$

Current practice according to FHWA-RD-97-130 is to use a minimum factor of safety of 1.5 against basal heave. A more detailed discussion is presented in Sections 54.3 and 37.3.1 of Terzaghi, Peck, and Mesri (1996). Cacoilo, Tamaro, and Edinger (1998) indicate a minimum safety factor of 1.5 is desirable to limit soil displacements. FHWA-SA-99-015 (page 107) notes that as the factor of safety decreases, loads on the lowest anchor increase. FHWA-SA-99-015 suggests a minimum factor of safety against basal heave of 2.5 for permanent facilities and 1.5 for support excavation facilities. Factors of safety below these target values indicate that more rigorous procedures such as limit equilibrium methods or Henkel's method should be used to evaluate design earth pressure loadings according to FHWA-SA-99-015 (page 107).

For weak soils (soft clays and loose sands) the failure surface will extend below the bottom of the cut, rather than through the bottom corner of the cut (FHWA-RD-98-065). For clay soils the deeper failure plane condition (i.e., below the bottom of the cut condition) can be evaluated by the Bishop method. The Bishop method is in reasonable agreement with the Henkel method (FHWA-SA-99-015). The Bishop method can be used in a GPSSP to determine the total load the tieback system must carry to meet the factor of safety requirements established for the project. The total load determined from a Bishop method internal stability limiting equilibrium analysis can be redistributed into an apparent pressure diagram. This apparent pressure diagram should be used as a basis for design if it provides a greater total load than that obtained from either a conventional apparent pressure diagram that assumes a “bottom corner of the cut” failure condition, or from an apparent pressure diagram constructed for the drained (long-term) condition. GPSSP analyses are described in FHWA-RD-98-065 and in Strom and Ebeling (2002b). GPSSP analyses should always be used to verify that the total load required to meet internal stability safety requirements is equal to or less than that used for the original design.

The calculated mass stability (i.e., external stability) slip circles for soft clays can be deep, and are generally located beyond or at the end of the tieback anchorage zone (Cacoilo, Tamaro, and Edinger 1998). Possibly, ground mass stability can be improved by increasing the depth of the sheeting or by extending the length of the tiebacks, although significant improvement in the factor of safety can sometimes be difficult to obtain by these methods (Cacoilo, Tamaro, and Edinger 1998).

4.2.12 Summary of results for “safety with economy” design

a. AZ 13 hot-rolled, Grade 50, sheet piles.

b. Four 1-3/8 in.-diameter, 150 Grade bar tendon at 8.8-ft spacing for anchor.

c. Two C12×30 wales, Grade 50 for all rows of post-tensioned tieback anchors.

d. 6-in. × 6-in. × 2-1/2-in. thrust plate.

e. 9-ft toe penetration depth.

4.3 “Stringent Displacement Control” Design Approach

For the Corps’ “stringent displacement control” design, a limiting equilibrium approach is used with a factor of safety of 1.5 applied to the shear strength of the soil. The total earth pressure load (P_{tl}) is then determined based on the limiting equilibrium analysis. Limiting equilibrium calculations for the “stringent displacement control” design are provided below. This process produces an EPF equal to 26.0 pcf, compared with an EPF of 22.7 pcf determined by the previous limiting equilibrium analysis for the “safety with economy” design (Section 4.2) using drained strength parameters (i.e., long-term loading condition). The total earth pressure load is determined assuming the shear strength of the soil is factored by the target factor of safety such that

$$\phi_{mob} = \tan^{-1}(\tan\phi / FS)$$

and

$$\phi_{mob} = \tan^{-1}(\frac{\tan\phi}{1.5})$$

$$= \tan^{-1}(\frac{\tan 36°}{1.5}) = 25.8$$

$$K_A = \tan^2\left(45° - \frac{\phi_{mob}}{2}\right) = 0.394$$

$$P_{tl} = K_A \gamma \frac{H^2}{2} = 0.394 * 132 * \frac{50^2}{2} = 65010 \text{ lb/ft}$$

$$\text{Effective pressure factor, EPF} = \frac{P_{tl}}{H^2} = 26 \text{ lb/ft}^3$$

An EPF value of 26 pcf is used in the construction of the apparent earth pressure diagram and in all subsequent computations of the prestress design anchor forces.

4.3.1 Anchor system

As noted in Section 1.5, a minimum of four rows of anchors is assumed. Further, the soil properties indicate stiff clay (see Section 3.2.1).

4.3.2 Anchor points

One of the intended purposes of installing a tieback wall is to restrict wall and retained soil movements during excavation to a tolerable movement so that adjacent structures will not experience any distress. If a settlement-sensitive structure is founded on the same soil used for supporting the anchors, a tolerable ground surface settlement may be less than 1/2 in. according to FHWA-RD-81-150. FHWA-RD-81-150 also states that if the adjacent structure has a deep foundation derives its capacity from a deep bearing stratum not influenced by the excavation activity, settlements of 1 in. or more may be acceptable. Obviously, this guidance is geared toward situations involving buildings that are adjacent to the excavation. Figure 75 in FHWA-RD-97-130 gives settlement profiles/envelopes behind flexible walls in different soils.

Wall and retained soil movements predictions are based on experience. Several types of movements are associated with flexible anchored walls. These are described on page 120 of FHWA-SA-99-015. Movement can occur due to (1) wall cantilever action associated with installation of the first anchor; (2) wall bulging actions associated with subsequent excavation stages and anchor installations; (3) wall settlement associated with mobilization of end bearing; (4) elastic elongation of the anchor tendons associated with a load increase; (5) anchor yielding or load redistribution in the anchor bond zone; and (6) mass ground movements behind the tieback anchors. The last three components of deformation result in translation of the wall and are generally small for anchored walls constructed in competent soils according to FHWA-SA-99-015. Typical lateral and horizontal movements for flexible retaining walls have been presented by Peck (1969), FHWA-RD-75-128, and Clough and O'Rourke (1990). FHWA-RD-97-130 states that maximum lateral movements in ground suitable for permanent ground anchor walls are generally less than 0.005H, with average maximum movements of about 0.002H. For a 50-ft-high wall the average maximum horizontal movement would be 1.2 in. by this relationship. FHWA-RD-97-130 also states that maximum vertical settlements in ground suitable for permanent ground anchor walls are generally less than 0.005H, with average maximum settlement tending toward 0.0015H. Maximum settlement occurs near the wall. For a 50-ft-high wall the average maximum settlement would be 0.9 in. by this relationship. Note that actual wall performance and especially horizontal and vertical deformations, are a function of both design and construction details.

Lateral wall movements and ground settlements cannot be eliminated for flexible tieback walls. However, they can be reduced by (1) controlling sheet-pile bending deformations (i.e., cantilever and bulging displacements); (2) minimizing sheet-pile settlements by installing the tieback anchors at flat angles (note that grouting of anchors installed at angles less than 10 degrees from horizontal is not common unless special grouting techniques are used) and properly designing the embedded portion of the wall to carry applied axial loads; and (3) increasing the magnitude of the anchor design forces for which the anchors are prestressed to over that obtained in a "safety with economy" design (given in Section 4.2).

Among the factors contributing to bending deformations are (1) the depth of excavation prior to installation and prestress of the first row of anchors, and (2) the span between the subsequent, lower rows of anchors. FHWA-RD-97-130 and others observe that reducing the distance to the upper ground anchor will reduce the cantilever bending deformations. The magnitude of this deformation, which occurs prior to installation of the first row of anchors, increases as the depth of excavation to the upper ground anchor increases. This deformation is often a significant contributor to total wall permanent deformations. Additional displacement constraints are invoked by reducing the span between the ground anchors, which will reduce the bulging deformations. The relationships developed by FHWA-RD-98-067 are recommended in a "displacement control" design procedure given in FHWA-RD-97-130 (page 147) and have been adopted for use in this report. Specifically, the FHWA-RD-97-130 Equation 9.1 is used to estimate cantilever displacement y_c, and Equation 9.2 is used to used to estimate bulging deformations y_b and will be given subsequently. The designer sets project-specific horizontal displacement limitations, which, in turn, are set as limiting values for y_c and y_b. The first-row anchor depth and spacings for the subsequent rows of anchors are then established that meet this project-specific displacement performance objective. A subsequent example calculation will demonstrate this procedure. On page 148 of FHWA-RD-97-130 the designer is cautioned that movements estimated from these two equations show trends, and they can be used to evaluate the impact of different ground anchor locations. They represent minimum movements that might be expected.

The third distinguishing aspect of the "stringent displacement control" design procedure is the factor of safety used in the EPF computation, set equal to 1.5 as compared with the 1.3 value used in the "safety with economy" design procedure. For this 50-ft-high wall problem, the EPF now becomes 26.0 pcf, which is 15 percent greater than the 22.7-pcf EPF value used in Section 4.2 "safety with economy" tieback wall design. Recall that the EPF value will scale the apparent earth pressure diagram used to compute the horizontal design anchor forces, designated as variable T_i in this report (where the subscript i designates the anchorage row number). It is inferred that by using a factor of safety equal to 1.5 in the development of apparent pressure diagram, nearer to at-rest conditions (versus active earth pressure conditions) will occur behind the wall, which along with smaller distance to upper ground anchor and closer anchor spacings, will contribute to reduce wall displacements over a "safety with economy" design. When displacement control of flexible tieback walls is a key consideration, the reader is referred to helpful discussions contained within Section 9.1 of FHWA-RD-97-130; Section 2.1.3 of FHWA-RD-81-150; Section 5.11.1 in FHWA-SA-99-015. It should be recognized, however, that where displacement is important to project performance, NLFEM-SSI analysis might be required to properly assess displacement performance. Additional information on NLFEM analysis can be found in Strom and Ebeling (2001). Alternatively, stiff tieback walls should always be considered in those situations where the magnitude of flexible tieback wall deformations (cantilever, bulging, and/or cumulative/final displacements) are of concern (see Strom and Ebeling 2001 or Strom and Ebeling 2002a).

Displacement limits are project specific. For this particular 50-ft-high wall design example, a maximum lateral wall displacement of 0.5 in. will be established for the FHWA-RD-98-067 cantilever displacement y_c and the bulging deformation y_b equations.

Using the empirical apparent earth pressure envelope (Figure 5.4, Strom and Ebeling 2001, and Figure 29, FHWA-RD-97-130), the vertical anchor intervals with four-tier anchoring for approximate balanced moments are determined.

$$\frac{1}{10} H^2_{(2,3,4,5)} = \frac{13}{54} H_1^{\ 2}$$

$$H_{(2,3,4,5)} = \sqrt{\frac{130}{54}} H_1 = 1.55 H_1$$

i.e.,

$$H = H_1 + H_2 + H_3 + H_4 + H_5 = H_1 + 4(1.55 H_1)$$

$$50 = 7.2 H_1$$

$$\therefore H_1 = 6.94 \text{ ft}$$

$$H_{(2,3,4,5)} = 1.55 * 6.94 = 10.757 \text{ ft}$$

Try $H_2 = H_3 = H_4 = H_5 = 10$ ft, 6 in. and $H_1 = 8$ ft, 0 in.

These anchor spacings will be evaluated to determine if the associated cantilever and interior spans can be used to meet stringent displacement control performance requirements.

Approximate cantilever deformation, y_c, allowing 1.5 ft overexcavation for placement of top anchor, $h_1 = 8 + 1.5 = 9.5$ ft and with $E_s = 2{,}850$ psi for stiff clay and $K_o = 0.5$,

$$y_c = \frac{4K_0 \gamma h_1^2}{E_s} = \frac{4 * 0.5 * 132 * 9.5^2}{2850 * 12} = 0.697 \text{ in.} > 0.5 \text{ in. NG}$$

The soil modulus (E_s) was obtained from Table 20 of FHWA-RD-97-130.

Approximate span bulging deformation, y_b, with h = 10.75 ft and wall height = 50 ft,

$$y_b = \frac{0.8K_0 \gamma hL}{E_s} = \frac{0.8*0.5*132*10.5*50}{2850*12} = 0.81 \text{ in.} > 0.5 \text{ in. NG}$$

Anchor spacing must be reduced to limit deformation to less than half an inch. Revise spacing using deformation constraints.

Cantilever deformation:

$$y_c = \frac{4K_0 \gamma h_1^2}{E_s} = \frac{4 * 0.5 * 132 * h_1^2}{2850 * 12} = 0.5$$

$$h_1 = 8.05 \text{ ft,}$$

Allowing 1.5 ft below anchor point, $H_1 = 8.05 - 1.5 = 6.54$ ft.

Span bulging deformation:

$$y_b = \frac{0.8K_0 \gamma hL}{E_s} = \frac{0.8 * 0.5 * 132 * h * 50}{2850 * 12} = 0.5$$

$$h = 6.47 \text{ ft}$$

Try an eight-tier anchor system with $H_1 = 6$ ft 3 in. and $H_2 = H_3 = H_4 = H_5 = H_6 = H_7 = H_8 = 6$ ft 3 in.

Anchor spacing satisfies the cantilever and bulging deformation constraints of not greater than 0.5 in. by the Mueller et al. (1996) equations. Note that no constraints on total (i.e., post-construction) horizontal and vertical wall deformations were considered in these computations. Recall that FHWA-RD-97-130 relationships for average maximum horizontal displacements and average maximum settlement (assuming good construction practice in conjunction with good design) may be on the order of 1.2 in. and 0.9 in., respectively. For displacement-sensitive projects, NLFEM analysis of the flexible wall is recommended. Alternatively, a stiff tieback wall system may be considered.

4.3.3 Apparent earth pressure

The effective earth pressure (p_e) is

$$p_e = \frac{P_{tl}}{H - \frac{H_1}{3} - \frac{H_8}{3}}$$

$$= \frac{65010}{50 - \frac{6.25}{3} - \frac{6.25}{3}} = 1418 \text{ lb/ft/ft}$$

4.3.4 Horizontal components of anchor loads

From Figure 5.4b (Strom and Ebeling 2001), the horizontal component of each anchor load T_i is determined. Anchor loads are expressed in pounds per foot run of wall.

$$T_1 = \left(\frac{2}{3}H_1 + \frac{1}{2}H_2\right)p = \left(\frac{2}{3} * 6.25 + \frac{1}{2} * 6.25\right) * 1418 = 10340 \text{ lb/ft}$$

$$T_{(2,3,4,5,6,)} = \left(\frac{H_2}{2} + \frac{H_3}{2}\right)p = \left(\frac{6.25}{2} + \frac{6.25}{2}\right) * 1418 = 8863 \text{ lb/ft}$$

$$T_7 = \left(\frac{H_7}{2} + \frac{23}{48}H_8\right)p = \left(\frac{6.25}{2} + \frac{23}{48} * 6.25\right) * 1418 = 8678 \text{ lb/ft}$$

4.3.5 Anchor loads (TF)

For constructibility, an anchor inclination of 10 degrees to the horizontal will be used and the total anchor force (TF) per foot run of wall determined. Assumed anchor spacing = 8.8 ft.

Top tier:

$$TF_1 = \frac{T_1}{\cos 10^0} = \frac{10340}{\cos 10^0} = 10500 \text{ lb/ft}$$

(Design anchor force = 10.5 kips/ft × 8.8 ft = 92.4 kips)

Tiers 2, 3, 4, 5, 6:

$$TF_{(2,3,4,5,6)} = \frac{T_{(2,3,4,5,6)}}{\cos 10^0} = \frac{8863}{\cos 10^0} = 9000 \text{ lb/ft}$$

(Design anchor force = 9 kips/ft × 8.8 ft = 79.2 kips)

Lower tier:

$$TF_7 = \frac{T_7}{\cos 10^0} = \frac{8678}{\cos 10^0} = 8812 \text{ lb/ft}$$

(Design anchor force = 8.812 kips/ft × 8.8 ft = 77.5 kips)

<u>Use</u> TF = 10,500 lb/ft for anchor design.

4.3.6 Subgrade reaction using tributary method

From Figure 5.4b (Strom and Ebeling 2001), the subgrade reaction (*R*) is determined. The subgrade reaction is expressed in pounds per foot run of wall.

$$R = \left(\frac{3}{16} H_8\right) p = \left(\frac{3}{16} * 6.25\right) * 1418 = 1662 \text{ lb/ft}$$

4.3.7 Bending moments

Using the information contained in Figure 5.4b of Strom and Ebeling (2001), the cantilever moment (M_1) and the maximum interior span moments (MM_1) can be determined. Moments are per foot run of wall.

Negative moment at point of top anchor is:

$$M_1 = \frac{13}{54} H_1^{\,2} p = \frac{13}{54} * 6.25^2 * 1418 = 13335 \text{ ft} - \text{lb/ft}$$

Maximum moment below top tier anchor:

$$MM_{(1,2,3,4,5,6,7)} = \frac{1}{10} H^2{}_{\max}\, p\,, \quad \begin{pmatrix} H_{max} \text{ is the larger of } H_2, H_3, \\ H_4, H_5, H_6, H_7, H_8 \end{pmatrix}$$

$$= \frac{1}{10} * 6.25^2 * 1418 = 5539 \text{ ft} - \text{lb/ft}$$

NOTE: Moments are not balanced because anchor spacing is determined by stringent displacement design criterion.

USE design moment M = 13335 ft-lb/ft.

4.3.8 Design of vertical sheet-pile system components

4.3.8.1 Select economical sheet-pile section. In accordance with Corps criteria (HQUSACE 1994), the allowable stresses for the sheet piling and wales shall be as follows:

Bending (i.e., combined bending and axial load): $f_b = 0.5\, f_y$

Shear: $f_v = 0.33\, f_y$

Allowable stresses are based on 5/6 of the AISC-ASD recommended values (AISC 1989) and reflect the Corps' design requirements for steel structures.

Try AZ 13 ARBED hot-rolled Grade 50 sheet pile with

section modulus about bending axis, $S_x = 24.2$ in.^{3}/ft

width per sheet, $w = 26.38$ in.

Moment on sheet pile per foot width = 13335 lb/ft

Assuming allowable bending stress for Grade 50 steel $f_b = 25$ ksi

Assuming allowing shear stress for Grade 50 steel $f_v = 16.5$ ksi

Required section modulus = 13.335*12/25 = 6.4 in.^{3} < 24.2 in.^{3} OK

Check shear capacity:

Maximum shear force, $V_{max} = T_{max} = 13.34$ kips/ft

$$\text{Required area, } A = \frac{13.34}{16.5} = 0.81\,\text{in.}^2 \text{ per ft run}$$

Shear area provided by an AZ 13 (Equation 6-5 in EM 1110-2-2504 (HQUSACE 1994))

$$= \frac{t_w \bullet h}{w} = \frac{0.375\,\text{in.} \bullet 11.93\,\text{in.}}{26.38\,\text{in.} \bullet \dfrac{\text{ft}}{12\,\text{in.}}}$$

$$= 2.04\,\text{in.}^2 \text{ per ft run } > 0.81\,\text{in.}^2 \text{ per ft run} \qquad \text{OK}$$

where

t_w = thickness of the web portion of the Z = 0.375 in.

h = height of the Z = 11.93 in.

Use AZ 13 Grade 50 sheetpiling.

4.3.8.2 Select economical bar tendon. A 150 grade prestressing steel bar will be selected from Table 8.4 of Strom and Ebeling (2001) to meet "stringent displacement control" design requirements. Anchors will be spaced to occur at the center of every fourth pair of *z*-section sheet piling (i.e., anchor spacing = 4 (26.38) = 105.52 in. = 8.8 ft). It is assumed that the final anchor prestress force (after losses) will equal 0.6 f_{pu} A_{ps}, where:

f_{pu} = anchor ultimate tensile strength = 150 ksi

A_{ps} = Cross-sectional area of bar tendon (in.^{2})

Bar tendons, rather than wire-strand tendons, are used to facilitate construction of the sheet pile-wale-anchor system. Details of this system are similar to those illustrated in Figure 4.1 for the "safety with economy" design.

As per the "safety with economy" design, a 7.5-in. diameter cased borehole will be used to place the bar tendons. The casing will be pulled as grouting takes place. Bar tendons are over 60 ft long, so a coupler will be needed.

Total anchor load used for the design of the wales and thrust plates at each tier level will be based on that determined for the top tier since it has the greatest total load:

(TL) = 10,500 (8.8) = 92,400 lb = 92.4 kips

From Table 8.4 of Strom and Ebeling (2001) a 1-1/4-in.-diameter, 150 grade bar tendon at 0.6 f_{pu} A_{ps}, can provide a final prestressing force up to 112.5 kips > 92.4 kips OK

4.3.8.3 Select wales. The wales are positioned on the outside of the sheet pile as shown in Figure 4.1. The design moment for continuous wales can be approximated using Equation 6-14 of EM 1110-2-2504 (HQUSACE 1994):

$$M_{Max} = \frac{T_{ah} S^2}{10}$$

where

T_{ah} = anchor force per foot of wall = 92.4 ÷ 8.8 = 10.50 kips per foot of wall

S = distance between adjacent anchors = 8.8 ft

$$M_{Max} = \frac{T_{ah} S^2}{10} = \frac{10.50(8.8)^2}{10} = 81.31 \text{ ft-kips}$$

The allowable stress design provisions of AISC (1989) will be used in accordance with Corps criteria as specified in EM 1110-2-2504. As such allowable stresses, or allowable loads, will be 5/6 of the appropriate AISC-ASD requirement (AISC 1989).

Using 50 grade steel, the required section modulus (S_x) assuming an allowable bending stress of 5/6 × 0.60 F_y, or 0.5 (50 ksi) is:

$$S_x = \frac{81.31(12)}{25} = 39.03 \text{ in.}^3$$

Two C10×30 channels back-to-back have a section modulus of 2(20.7) = 41.4 in.^{3} > 39.03 in.^{3} OK. Space channels at 3.0-in. back-to-back per the Figure 4.1 details developed for the "safety with economy" design.

4.3.8.4 Select thrust (bearing) plate. Try a 6-in.-wide × 6-in.-long × 2.25-in.-deep thrust plate. Details are similar to those of Figure 4.2 for the "safety with economy" design.

The bearing pressure (w) exerted by the thrust plate on each wale is:

$$w = \frac{TL}{2Bn} = \frac{92.4}{2(6)1.5} = 5.133 \text{ kips per in.}^2$$

where

B = bearing plate width = 6 in.

n = bearing contact width on each wale = 1.5 in.

TL = total anchor load = 92.4 kips

The maximum moment on the plate (M_{PL}) is:

$$M_{PL} = \frac{TL(S)}{4} = \frac{92.4(4.5)}{4} = 103.95 \text{ in.-kips (See Figure 4.2b)}$$

Using 50 Grade steel, the required section modulus (S_x) assuming an allowable bending stress of 5/6 × 0.60 F_y, or 0.5 (50 ksi) is:

$$S_x = \frac{103.95}{25} = 4.16 \text{ in.}^3$$

Section modulus provided (S_{PL}) is:

$$S_{PL} = \frac{Bd^2}{6} = \frac{6(2.25)^2}{6} = 5.06 \text{ in.}^3 > 4.16 \text{ in.}^3 \text{ OK}$$

Use 6-in.-wide × 6-in.-long × 2.25-in.-deep thrust plate.

Checking shear in the thrust plate:

$$f_v = \frac{TL}{2Bd} = \frac{92.4}{(2)(6)(2.25)} = 3.42 \text{ ksi} < \frac{5}{6}\left(0.40\,F_y\right) = 16.67 \text{ ksi} \qquad \text{OK}$$

4.3.8.5 Check web yielding. In accordance with AISC-ASD Equation K-2 (AISC 1989) the maximum interior load reaction for web yielding (R) is:

$R = 5/6(0.66\,F_y)\,t_w\,(N + 5k)$

where

N = bearing length = 6 in.

k = distance from top flange surface to web toe of fillet

= 1.000 in. for C10×30 channel

t_w = web thickness = 0.673 in. for C10×30 channel

R = 0.55 (50) (0.673) [6 + 5(1.000)] = 203.6 kips > 62.75 kips OK

4.3.8.6 Check web crippling. In accordance with AISC-ASD Equation K1-4 (AISC 1989) the maximum concentrated load (L_{CR}) for web crippling is:

$$L_{CR} = 67.5 t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{F_y\left(\frac{t_f}{t_w}\right)}$$

where

d = overall depth of member = 10.00 in. for C10×30

t_f = flange thickness = 0.436 in. for C10×30 channel

Maximum concentrated load (L_{CR}) is proportional to stress. Since a 5/6 reduction is being used to obtain allowable stress, 5/6 of maximum web crippling load (L_{CR}) is used.

$$L_{CR} = \frac{5}{6}\left[67.5(0.673)^2\left[1+3\left(\frac{6}{10}\right)(1.544)^{1.5}\right]\sqrt{50(0.648)}\right]$$

$$= 645.8 \text{ kips} > 62.75 \text{ kips} \quad \text{OK}$$

4.3.8.7 Check web compression buckling. In accordance with AISC-ASD Equation K1-8 (AISC 1989) it can be determined whether or not web stiffeners are required to prevent compression buckling of the C10×30 channel web.

$$\frac{4100(t_w)^3\sqrt{F_y}}{P_{bf}} = \frac{4100(.673)^3\sqrt{50}}{\frac{5}{3}(46.2)} = 114.8 \text{ in.} < d_c = 8.00 \text{ in.}$$

OK - stiffeners not required.

where

P_{bf} = the computed force delivered by the flange (= 92.4/2) multiplied by 5/3.

d_c = d – 2k = 8.00 in. for C10×30 channel.

4.3.8.8 Check web sidesway buckling. The outside flanges of the C10×30 channels are to be welded to the thrust plate and the inside flanges are welded to a wedge plate that in turn is welded to the sheet piling. Details are similar to those of Figure 4.1 for the "safety with economy" design. With this construction detail, in conjunction with the use of a prestressed bar tendon, the C10×30 channels are likely to be braced against sidesway at the point of load application by the bar tendon (prestressed in tension), and sidesway buckling is not likely to occur. However, should sidesway be of concern to the designer, equation K1-6, given in AISC-ASD Chapter K, section K1, subsection 5 (AISC 1989) can be used to determine whether or not sidesway buckling is an issue for the loaded flange (of the C-channel) restrained against rotation. (Equation K1-7 is for a loaded flange not restrained against rotation.)

4.3.9 Anchor lengths

4.3.9.1 Unbonded anchor length, L_i. Assume 10-degree inclination for all anchors. The unbonded length must be sufficient such that anchor bond zone is beyond the short-term (undrained) and long-term (drained) potential failure surfaces and satisfy the Figure 8.5, Strom and Ebeling (2001), length criteria. With the short-term shear strength characterized in terms of S_u equal to 2,400 psf (with ϕ = 0 degree), and with the long-term shear strength characterized in terms of ϕ' equal to 36 degrees, the short-term loading condition will require greater unbonded anchor lengths. Thus, the potential failure plane will be based on the undrained shear strength with ϕ = 0 degree (as is also done in the FHWA-RD-97-130 cohesive design example; refer to Figure 107).

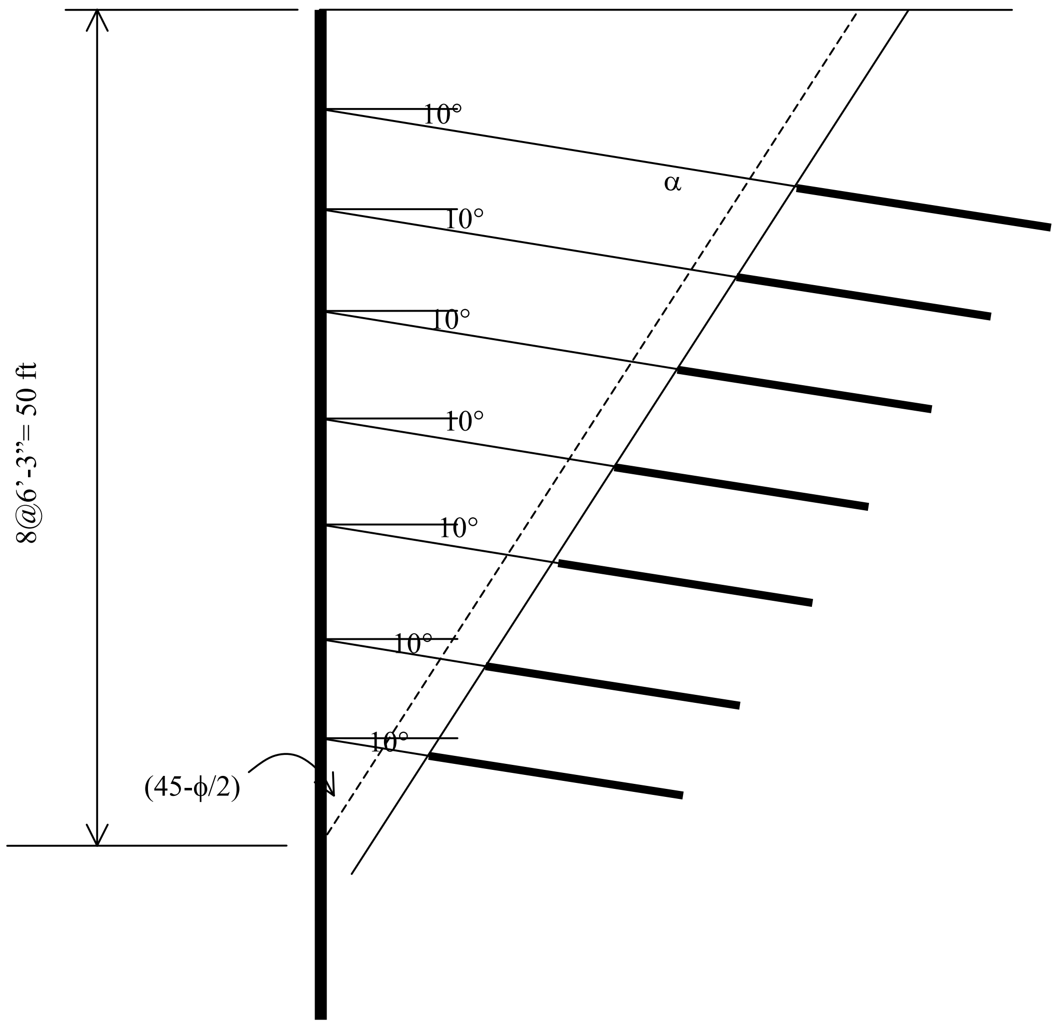


Figure 4.4. Seven-tier anchor

$$\frac{\text{unbonded length, } L}{(45° - \phi/2)} = \frac{\text{height of anchor point}}{\alpha} \qquad (\text{see Figure 4.4})$$

$$45° - \frac{\phi}{2} = 45° - \frac{0°}{2} = 45°$$

$$\alpha = 180° - 45° - 80° = 55°$$

Top-tier anchor:

$$\frac{L}{\sin 45} = \frac{43.75}{\sin 55}$$

$$L = \frac{43.75 * \sin 45}{\sin 55} = 37.77 \text{ ft}$$

Allowing 5 ft or 0.2H clearance beyond shear plane (see Figure 8.5, Strom and Ebeling 2001).

$$L_1 = 37.77 + (0.2\text{H or 5 ft, whichever is greater})$$
$$= 37.77 + 10 = 47.77 \text{ ft} \; > 10 \text{ ft minimum required for bar anchor} \quad \text{OK}$$

(Minimum required for strand anchor is 15 ft)

Similarly, second-tier anchor:

$$L_2 = \frac{37.5}{43.75} * L + 0.2H$$
$$= 42.4 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

Third-tier anchor:

$$L_3 = \frac{31.25}{43.75} * L + 0.2H$$
$$= 37 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

Fourth-tier anchor:

$$L_4 = \frac{25}{43.75} * L + 0.2H$$
$$= 31.6 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

Fifth-tier anchor:

$$L_5 = \frac{18.75}{43.75} * L + 0.2H$$
$$= 26.2 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

Sixth-tier anchor:

$$L_6 = \frac{12.5}{43.75} * L + 0.2H$$
$$= 20.8 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

Seventh-tier anchor:

$$L_7 = \frac{6.25}{43.75} * L + 0.2H$$
$$= 15.4 \text{ ft} > 15 \text{ ft} \quad \text{OK}$$

The unbonded length determined above should be verified using the internal stability analysis procedures described for both undrained (short-term) and drained (long-term) conditions in Strom and Ebeling (2002b). The verification process uses limiting equilibrium procedures, which can be performed by simple hand calculations or GPSS procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to meet internal stability performance requirements (i.e., factor of safety of 1.5 for a stringent displacement control design).

4.3.9.2 Bonded length of anchors, L_b. The usual practice is for the wall designer to specify the anchor capacity and any right-of-way and easement constraints required of the anchorage system. It is up to the tieback anchor contractor, usually a specialty subcontractor, to propose the type of anchorage system to be used to meet the wall design requirements. A preliminary estimate of the bond length (L_b) required to develop the ground anchors is provided below.

The horizontal anchor force T_1 corresponds to maximum horizontal anchor force T_{max} (Section 4.3.4). Because the horizontal anchor forces T_2, T_3 T_4, T_5, and T_6 are within 14 percent of this T_{max} value and T_7 is within 17 percent of this T_{max} value, the bond length computations will be made using the tendon force value of T_{max}. The computed bond length will be slightly conservative for anchor tendons 2 through 7.

Top tier:

With 8.8-ft horizontal spacing between anchors the maximum anchor (tendon) force Al = T_1 * 8.8 = 10,500*8.8 = 92,400 1b = 92.4 kips

An empirical method (Equation 1.2) was attempted to estimate the bond length of large-diameter straight-shaft gravity-grouted anchors for preliminary design purposes. Recall that it is up to the tieback anchor contractor, usually a specialty subcontractor, to propose the type of anchorage system to be used to meet the wall design requirements.

The design tendon force for the anchor bond zone is computed equal to 92.4 kips. Applying a factor of safety equal to 2.0 to this design force results in an ultimate anchor force equal to 184.8 kips.

No site-specific anchor load test data are available for use in this design. Consequently, preliminary design computations are made using traditional (non-site specific) assumptions for the range in value of the adhesion factor when computing an average ultimate soil-to-grout bond stress (Equation 1.3) and subsequently, the ultimate capacity of a large-diameter straight-shaft gravity-grouted anchor of 40-ft length (the maximum possible length without requiring specialized methods). The ultimate anchor capacity of a large-diameter straight-shaft gravity-grouted anchor was computed using Equation 1.2. These computations (not shown) indicate that a more robust anchorage system will be required to achieve an ultimate anchorage capacity of 184.8 kips.

Review of information contained in Section 1.5.4 indicates that a post-grouted (regroutable) ground anchor is a possible solution. (The anchor capacities cited for the case histories given and for the soil conditions cited in this section makes this type of anchorage system a viable candidate.) For an ultimate anchor force equal to 184.8 kips and a assuming a 40-ft bond length, the ultimate capacity of rate of load transfer corresponds to 4.62 kips per lineal ft. A preliminary bonded length L_b of 40 ft will be assumed for anchorage layout purposes. Recognize that the final bond length, anchor capacity, etc., will be confirmed by proof-testing and performance testing onsite (Strom and Ebeling 2002b).

Total anchor lengths ($Lt_i = L_i + L_b$).

Top-tier anchor:

$$Lt_1 = 47.8 + 40 = 87.8 \text{ ft} \approx 88 \text{ ft}$$

Second-tier anchor:

$$Lt_2 = 42.4 + 40 = 82.4 \text{ ft} \approx 83 \text{ ft}$$

Third-tier anchor:

$$Lt_3 = 37 + 40 = 77 \text{ ft}$$

Fourth-tier anchor:

$$Lt_4 = 31.6 + 40 = 71.6 \text{ ft} \approx 72 \text{ ft}$$

Fifth-tier anchor:

$$Lt_5 = 26.2 + 40 = 66.2 \text{ ft} \approx 67 \text{ ft}$$

Sixth-tier anchor:

$$Lt_6 = 20.8 + 40 = 60.8 \text{ ft} \approx 61 \text{ ft}$$

Seventh-tier anchor:

$$Lt_7 = 15.4 + 40 = 55.4 \text{ ft} \approx 56 \text{ ft}$$

The total anchor length (bonded + unbonded length) determined above should be verified using the external stability analysis procedures described in Strom and Ebeling (2002b) for both short-term (undrained) and long-term (drained) conditions. This verification process also uses limiting equilibrium procedures, which can be performed by simple hand calculations or GPSS procedures. The verification process ensures that the anchorage is located a sufficient distance behind the wall to prevent ground mass stability failure (i.e., meet external stability performance requirements with factor of safety of 1.5 for a "stringent displacement control" design).

4.3.10 Determine required depth of sheet pile penetration, D

Passive resistance mobilized in front of the toe must be adequate to resist the reaction with a factor of safety of 1.5 (Section 8.7.1 of Strom and Ebeling 2001; Section 6.2 of FHWA-RD-97-103).

4.3.10.1 Short-term undrained condition. For a continuous wall, the passive earth pressure force, P_p, per ft run of wall is

$$P_p = \left(2 \bullet S_u + \gamma \bullet D + 2 \bullet S_u\right) \bullet \frac{D}{2} = \left(2 \bullet S_u \bullet D + \frac{\gamma \bullet D^2}{2}\right)$$

According to FHWA-RD-97-130, page 104, the Rankine active pressures must be applied to the other side (i.e., the retained side) of the wall when computing the capacity of the toe. Thus, the usable net resistance is given by

$$P_p - P_a = R \bullet FS$$

where the factor of safety, FS, is set equal to 1.5 (Section 8.7.1 of Strom and Ebeling 2001; Section 6.2 in FHWA-RD-97-130), R is equal to 1.662 kips/ft (Section 4.3.6), and the active force, P_a, per ft run of wall is

$$P_a = \left(\gamma \bullet H - 2 \bullet S_u + \gamma \bullet (H + D) - 2 \bullet S_u\right) \bullet \frac{D}{2}$$

$$= \left[\left(\gamma \bullet H - 2 \bullet S_u\right) \bullet D + \frac{\gamma \bullet D^2}{2}\right]$$

The net usable resistance to (*R*)(*FS*) becomes

$$P_p - P_a = 4 \bullet S_u \bullet D - \gamma \bullet H \bullet D$$

Solve for D by setting the net usable passive resistance equal to the factored reaction force (*R*)(*FS*).

$$4 \bullet S_u \bullet D - \gamma \bullet H \bullet D = R \bullet FS$$

$$4 \bullet 2.4 \bullet D - 0.132 \bullet 50 \bullet D = 1.662 \bullet 1.5$$

$$9.6 \bullet D - 6.6 \bullet D = 1.662 \bullet 1.5$$

$$3 \bullet D = 2.493$$

$$D = 0.83 \text{ ft}$$

4.3.10.2 Long-term drained condition. For a continuous wall, the passive earth pressure force, P_p, per ft run of wall is

$$P_p = K_p \bullet \gamma \bullet \frac{D^2}{2}$$

where conservatively assuming zero soil-to-steel interface friction, *Kp* by the Rankine relationship is

$$K_p = \tan^2\left(45° + \frac{\phi'}{2}\right) = \tan^2\left(45° + \frac{36°}{2}\right) = \tan^2(63°) = 3.85$$

According to FHWA-RD-97-130, page 104, the Rankine active pressures must be applied to the other side (i.e., the retained side) of the wall when computing the capacity of the toe. Thus, the usable net resistance is given by

$$P_p - P_a = R \bullet FS$$

where the factor of safety, *FS*, is set equal to 1.5 (Section 8.7.1 of Strom and Ebeling 2001; Section 6.2 in FHWA-RD-97-130), *R* is equal to 1.662 kips/ft (Section 4.3.6), and the active force, P_a, per ft run of wall is

$$P_a = K_a \bullet [\gamma \bullet H + \gamma \bullet (H + D)] \bullet \frac{D}{2}$$

$$= K_a \bullet \left[(\gamma \bullet H) \bullet D + \frac{\gamma \bullet D^2}{2}\right]$$

Assuming zero soil-to-steel interface friction, K_a (by the Rankine relationship), is

$$K_a = \tan^2\left(45° - \frac{\phi'}{2}\right) = \tan^2\left(45° - \frac{36°}{2}\right) = \tan^2(27°) = 0.26$$

The net usable resistance to (R)(FS) becomes

$$P_p - P_a = K_p \bullet \gamma \bullet \frac{D^2}{2} - K_a \bullet \left[(\gamma \bullet H) \bullet D + \frac{\gamma \bullet D^2}{2}\right]$$

Solve for D by setting the net usable passive resistance equal to the factored reaction force, (*R*)(*FS*):

$$K_p \bullet \gamma \bullet \frac{D^2}{2} - K_a \bullet \left[(\gamma \bullet H) \bullet D + \frac{\gamma \bullet D^2}{2} \right] = R \bullet FS$$

$$(K_p - K_a) \bullet \gamma \bullet \frac{D^2}{2} - K_a \bullet (\gamma \bullet H) \bullet D = R \bullet FS$$

$$(3.85 - 0.26) \bullet 0.132 \bullet \frac{D^2}{2} - 0.26 \bullet (0.132 \bullet 50) \bullet D = 1.662 \bullet 1.5$$

$$0.2369 \bullet D^2 - 1.716 \bullet D - 2.493 = 0$$

The formula for the solution of a quadratic equation for D is

$$D = \frac{-(-1.716) \pm \sqrt{(1.716)^2 - 4 \bullet (0.2369) \bullet (-2.493)}}{2 \bullet (0.2369)}$$

$$= \frac{1.716 \pm \sqrt{2.9447 + 2.3624}}{0.4738} = \frac{1.716 \pm 2.3037}{0.4738} = 8.48 \text{ ft}$$

Use $D = 9$ ft penetration.

4.3.10.3 Sheet-pile toe embedment. Sheet-pile toe embedment requirements for both vertical and horizontal loads must be determined. With respect to the vertical component of prestress anchor load:

$$\sum V = (V_1 + V_2 + V_3 + V_4 + V_5 + V_6 + V_7)$$

$$\sum V = (T_1 + T_2 + T_3 + T_4 + T_5 + T_6 + T_7) \bullet \tan(10°) \bullet 8.8$$

$$\sum V = (10340 + 5 \bullet (8863) + 8678) \bullet \tan(10°) \bullet 8.8$$

$$\sum V = 98{,}272.4 \text{ lb} = 98.27 \text{ kips}$$

The anchors are spaced at 8.8-ft intervals.

The following computations are made to determine total force that the sheet-pile foundation must resist. A 9-ft depth of penetration is assumed in these computations for a 50-ft exposed wall height.

The total sheet-pile and wale weight assuming a 9-ft toe length is equal to the vertical component of anchor force plus the weight of the sheet-pile plus the weight of fourteen MC 10×30 channels used to form the seven wales. The axial load transfer from the drilled shaft above the bottom of the wall to the retained soil, which acts upward on the sheet pile, is also included in the computations. The magnitude of each of these forces are summarized in the following steps:

a. Vertical component of anchor force = 98.27 kips, with a 8.8-ft anchor spacing. The vertical component of anchor force per ft run of wall = 11.17 kips per ft run of wall.

b. Computation of the axial load transfer from the sheet pile above the bottom of the wall to the retained soil:

(1) ASCE Geotechnical Special Publication No. 74 by a Committee on Earth Retaining Structures states on page 108 (ASCE 1997) that since nonhorizontal tiebacks exert a downward (anchor) force on the wall (through the wales), that tiebacks with modest inclinations are usually preferable to steep ones. They also note that when the tieback wall settles, less horizontal movement occurs with flatter tiebacks. Lastly, vertical effects are minimized if the sheet-pile wall is adequate to transmit the vertical loads to soil beneath the excavation, or if shear between the back of the sheeting and retained soil is adequate to provided the required vertical reaction.

(2) Axial load and ground movements are interrelated. The magnitude of the axial load depends upon the vertical components of the ground anchor loads, the strength of the supported ground, vertical and lateral movements of the wall, the relative movements of the ground with respect to the wall, and the axial load carrying capacity of the toe. FHWA-RD-98-066 (page 66) discusses results taken from walls in dense sands and stiff to hard clays in which the axial load measured in the soldier beam toes was less than the vertical components of the ground anchors. This favorable axial load transfer from the soldier beam to the retained soil is idealized in Figure 41(b) of FHWA-RD-97-130. Axial load transferred to the ground above the bottom of the excavations in stiff to hard clays was equal to A_s times ($0.25S_u$) according to FHWA-RD-97-130 (page 88). A_s was the surface area of the soldier beam in contact with the ground above the bottom of the excavation, and $0.25S_u$ was the back-calculated adhesion. At the stiff cohesive sites, the load transferred from the soldier beam to the ground above the bottom of the excavation appears to be valid for the long-term condition. Using an adhesion equal to 25 percent of the undrained shear strength gives a lower load transfer rate than a rate based on drained shear strengths. In adapting this to sheet-pile and wale retaining wall system, the subsequent calculations are made on a per ft run of wall basis. This computation assumes composite wall action for the sheet-pile and wale wall system.

(3) To take advantage of this axial load transfer from the sheet pile to the retained stiff to hard (cohesive) soil, Table 11 in FHWA-RD-98-066 and Table 11 in FHWA-RD-97-130 stipulate that

$$S_u > \frac{\gamma \bullet H}{4} - 5.714 \bullet H$$

which for this problem becomes

$$2{,}400 \text{ psf} > \frac{132 \text{ pcf} \bullet 50 \text{ ft}}{4} - 5.714 \bullet 50 \text{ ft}$$

$$2{,}400 > 1{,}650 - 285.7$$

$$2{,}400 > 1{,}364.3 \qquad OK$$

(4) So the following set of computations assumes that the axial load is transferred to the ground above the bottom of the excavation in this stiff clay site. Page 209 in FHWA-RD-97-130 gives this transfer force as

$$\text{Axial load transfer} = \alpha \bullet S_u \bullet A_s \bullet (H - H_1)$$

where H is the height of the wall (= 50 ft) and H_1 is the depth to the first row of anchors (6.25 ft in Section 4.3.2). A_s is approximated as equal to 1 ft for a continuous, in-plan, sheet-pile system (rather than A_s equal to half the circumference of the drilled-in soldier beam shaft as used in the Section 3.3.11 computations).

$$\begin{aligned}\text{Axial load transfer} &= 0.25 \bullet 2400 \text{ psf} \bullet 1 \text{ ft} \bullet (50 \text{ ft} - 6.25 \text{ ft}) \\ &= 600 \text{ psf} \bullet 1 \text{ ft} \bullet 43.75 \text{ ft} = 26{,}250 \text{ lb} \\ &= 26.25 \text{ kips per ft run of wall}\end{aligned}$$

(5) Note that this 26.25-kip force acts to reduce the axial load acting on the sheet-pile foundation. The magnitude of this upward-acting force (from the perspective of the sheet pile) is significant. Great care must be exercised by the designers when taking advantage of this load transfer mechanism. It is assumed in this wall design that the soldier beam wall settles relative to the ground. Before applying this force in a design, designers should review the discussion and guidance given on pages 87-90 of FHWA-RD-97-130 and pages 66-69 in FHWA-RD-98-066. The instrumented wall case histories are discussed in FHWA-RD-98-066.

(6) It is important to recognize also that when these computations are made on a per foot run of wall basis, it is assumed that the entire sheet-pile wall (consisting of four AZ 13 sheets between each column of tieback anchors and the rows of wales) acts as a composite wall system. For this to occur, all of the sheeting between tieback anchors must contribute to the (upward-acting) vertical shear force provided by the retained soil as well as resist the vertical component of the anchor forces. One means of accomplishing this (vertical shear transfer between sheets) is the use of welded wedge plate connections between each sheet pile and each wale (see Figure 4.1a). A vertical shear transfer capacity calculation should be

made by the wall designers to verify this composite wall assumption (calculations are not included in this example).

c. Weight of AZ 13 sheet-pile per ft run of wall = 59 ft * 0.02192 = 1.29 kips per ft run of wall.

d. Weight of 14 MC 10×30 channels for the seven rows of wales = 14*0.03 = 0.42 kips per ft run of wall.

e. Computation of the applied total axial load per ft run of wall:

$$\mathrm{Q}_{applied} = \sum V \text{ - Axial load transfer} + \text{Weight of sheet} - \text{piles} + \text{Weight of channels}$$

$$\mathrm{Q}_{applied} = 11.17 \text{ kips - } 26.25 \text{ kips} + 1.29 \text{ kips} + 0.42 \text{ kips} = -13.37 \text{ kips per ft run of wall}$$

Thus, for the continuous sheet-pile and wale retaining wall system with a 9-ft depth of penetration, there is no net downward applied axial load (as calculated on a per ft run of wall basis) due to the load transfer to the backfill.

4.3.11 Basal stability

The apparent pressure diagram used to compute the horizontal components of the anchor forces and the horizontal subgrade reaction force is for the condition where the soil at the bottom of the wall is not near a state of plastic equilibrium (i.e., failure), as discussed in Section 4.2.2 of FHWA-RD-97-130 and Section 5.8.2 of FHWA-SA-99-015. For a wide, infinitely long, excavation in a homogenous soft to medium clay of constant shear strength, the factor of safety is given by

$$FS = \frac{N_c}{\gamma \bullet \frac{H}{S_u}} = \frac{5.14}{N_s}$$

where

γ = total unit weight

N_s = stability number

Recall the stability number N_s has been used to identify excavation support systems with potential for movement and basal heave problems (Table 8.7 in Strom and Ebeling 2001; Table 12 in FHWA-SA-99-015). In this problem N_s is

$$N_s = \gamma \bullet \frac{H}{S_u} = 0.132 \bullet \frac{50}{2.4} = 2.75$$

Small values of N_s, with respect to a value of 5.14, indicate basal stability and small ground movements. The factor of safety against basal heave is

$$FS = \frac{5.14}{N_s} = \frac{5.14}{2.75} = 1.87$$

Current practice according to FHWA-RD-97-130 is to use a minimum factor of safety of 1.5 against basal heave. A more detailed discussion is presented in Sections 54.3 and 37.3.1 of Terzaghi, Peck, and Mesri (1996). Cacoilo, Tamaro, and Edinger (1998) indicate a minimum safety factor of 1.5 is desirable to limit soil displacements. FHWA-SA-99-015 (page 107) notes that as the factor of safety decreases, loads on the lowest anchor increase. FHWA-SA-99-015 suggests a minimum factor of safety against basal heave of 2.5 for permanent facilities and 1.5 for support excavation facilities. Factors of safety below these target values indicate that more rigorous procedures such as limit equilibrium methods or Henkel's method should be used to evaluate design earth pressure loadings according to FHWA-SA-99-015 (page 107).

For weak soils (soft clays and loose sands) the failure surface will extend below the bottom of the cut, rather than through the bottom corner of the cut (FHWA-RD-98-065). For clay soils the deeper failure plane condition (i.e., below the bottom of the cut condition) can be evaluated by the Bishop method. The Bishop method is in reasonable agreement with the Henkel method (FHWA-SA-99-015). The Bishop method can be used in a GPSSP to determine the total load the tieback system must carry to meet the factor of safety requirements established for the project. The total load determined from a Bishop method internal stability limiting equilibrium analysis can be redistributed into an apparent pressure diagram. This apparent pressure diagram should be used as a basis for design if it provides a greater total load than that obtained from either a conventional apparent pressure diagram that assumes a "bottom corner of the cut" failure condition, or from an apparent pressure diagram constructed for the drained (long-term) condition. GPSSP analyses are described in FHWA-RD-98-065 and in Strom and Ebeling (2002b). GPSSP analyses should always be used to verify that the total load required to meet internal stability safety requirements is equal to or less than that used for the original design.

The calculated mass stability (i.e., external stability) slip circles for soft clays can be deep, and are generally located beyond or at the end of the tieback anchorage zone (Cacoilo, Tamaro, and Edinger 1998). Possibly, ground mass stability can be improved by increasing the depth of the sheeting or by extending the length of the tiebacks, although significant improvement in the factor of safety can sometimes be difficult to obtain by these methods (Cacoilo, Tamaro, and Edinger 1998).

4.3.12 Summary of results for “stringent displacement control” design

a. AZ 13 hot rolled, Grade 50 sheet piles.

b. Seven 1-1/4-in.-diameter, 150 Grade bar tendon anchor at 8.8-ft spacing.

c. Two C10×30 wales, Grade 50, for all rows of post-tensioned tieback anchors.

d. 6-in. × 6-in. × 2-1/4-in. thrust plate.

e. 9-ft toe penetration depth.

5 Summary of Results for 25- and 35-ft-High Wall Systems

Two flexible tieback wall systems (with post-tensioned tieback anchors) commonly used in the construction of Corps of Engineers navigation projects were considered in this study:

- Soldier beam system with wood lagging, post-tensioned tieback anchors, and a permanent concrete facing. (Design of the permanent concrete facing was not included in this report.)

- Vertical sheet-pile system with wales and post-tensioned tieback anchors.

The purposes of the study were to

- Demonstrate the RIGID 1 Method, as applied to the design of flexible tieback wall systems with a horizontal retained soil surface.

- Compare the design results for walls heights of 25, 35, and 50 ft for a “safety with economy” performance objective and a “stringent displacement control” performance objective.

- Prepare study walls to be subjected to additional research using advanced, nonlinear finite element-based, SSI methods of analysis; i.e., a complete construction sequence analysis using PC-SOILSTRUCT-ALPHA of the top-down construction of these tieback retaining walls.

5.1 RIGID 1 Method

The RIGID 1 Method, described in Section 1.2 of the report, was applied to design the two flexible tieback wall systems, for both a granular soil site and a cohesive soil site. The apparent pressure loading used in the example problems is in accordance with FHWA-RD-97-130 (Figure 29). This information is also presented in Strom and Ebeling (2001; Figure 5.4).

When tiebacks are prestressed to levels nearer to active pressure conditions (versus at-rest conditions), the "total load" used to determine the apparent earth pressure is based on that approximately corresponding to a factor of safety of 1.3 on the shear strength of the soil. This total load approach was used for developing apparent pressure diagrams for those flexible wall systems required to meet "safety with economy" performance objectives.

When tiebacks are prestressed to minimize wall displacements, the total load used to determine the apparent earth pressure is based on use of an at-rest earth pressure coefficient, or that approximately corresponding to a factor of safety of 1.5 applied to the shear strength of the soil. (Refer to discussion in Section 1.1.2.2.) This total load approach was used for developing apparent pressure diagrams for those flexible wall systems required to meet "stringent displacement control" performance objectives.

For the "safety with economy" performance objective, it was assumed that the number of anchors used in each design would be consistent with past Corps practice. Therefore, two rows (i.e., tiers) of anchors were assumed for the 25-ft-high walls, three rows of anchors for the 35-ft-high walls, and four rows of anchors for the 50-ft-high walls.

With respect to the "stringent displacement control" performance objective, it was recognized that additional rows of anchors (i.e., reduced anchor spacing) might also be required to meet project performance objectives. The FHWA-RD-98-067 equation with a maximum wall displacement of 0.5 to 0.7 in. was established as the performance goal for the "stringent displacement control" designs, recognizing that displacement performance will be project specific. However, a maximum wall displacement of 0.5 in. in the FHWA-RD-98-067 equations was considered to be appropriate for those projects where settlement-sensitive structures are founded in close proximity to the tieback wall. Refer to Section 2.2.1 for more detailed discussions regarding displacements.

5.2 Results Comparison

The results for the various tieback wall heights (all with a horizontal retained soil surface) and performance objectives are summarized in the following section. Referring to Section 1.1.2, tieback wall system stiffness can be defined by EI/L^4, where EI is the stiffness of the wall and L is the distance between supports. For the most part, the number of rows of anchors (i.e., distance between supports) selected for the "safety with economy" design provided suitable tieback wall system stiffness for the "stringent displacement control" design, at least for the particular granular and cohesive soil sites selected for the comparison. For the granular soil site, all the tieback spacings used for the "safety with economy" designs were adequate for the "stringent displacement control" designs. This is also true with respect to the cohesive soil site for the 25- and 35-ft-high walls. However, for the 50-ft-high wall located at the cohesive (stiff clay) site, the number of tieback anchor rows (i.e., tiers) had to be increased from 4 to 7 rows to meet "stringent displacement control" performance objectives. For this study, the cantilever and bulging displacement demands on each wall were determined using equations developed by Mueller et al. (1998) (Equations 9.1 and 9.2 of FHWA-RD-97-130). This controlled the span arrangement (i.e., anchor rows and spacing) for the "stringent displacement control" designs.

The soil loadings for the "stringent displacement control" designs increased over those used for the "safety with economy" designs. This occurred due to an increase in the factor of safety applied to the shear strength of the soil (i.e., factor of safety of 1.3 for the "safety with economy" designs versus 1.5 for the "stringent displacement control" designs). As indicated above, for the 25- and 35-ft walls, the number of rows of anchors and the anchor spacings (in-plan) were the same for both the "safety with economy" and "stringent displacement control" designs. As a result, the increase in moment demand for the "stringent displacement control" designs, as compared with the "safety with economy designs," was for the most part proportional to the increase in soil loading. Moment demand comparisons for the soldier beam and sheet-pile wall systems are provided as the last two tables in the summary (Section 5.4).

5.3 Soil Properties

The granular soil site consists of loose sand and gravel. The properties used for the granular soil are

- Friction angle, ϕ = 30 deg.
- Unit weight, γ = 115 pcf.

The homogeneous cohesive soil site consists of stiff overconsolidated clay. The overconsolidation ratio for the soil is 3. The soil has a plastic limit of 19 percent. The soil properties used for the cohesive soil are in accordance with the "Cohesive Soil Example" (FHWA-RD-97-130, page 204):

- Undrained shear strength, S_u = 2,400 psf.
- Unit weight, γ = 132 pcf.
- Earth pressure factor for undrained (short-term) condition, EPF = 20 pcf.
- Friction angle for drained (long-term) condition ϕ = 36 deg. (Calculated via the Appendix A empirical correlation described in previous chapters.)

5.4 Summary

Summary results for 25- and 35-ft-high wall systems are presented in tabular form in this chapter for comparison. The calculations (not shown) follow those given in the previous chapters. For those anchor wall systems retaining stiff cohesive soil, the toe embedment is based on consideration of axial load transfer from the soldier beam to the retained soil in Tables 5.5 through 5.8, and consideration of axial load transfer from the sheet pile (with wales) to the retained soil in Tables 5.9 through 5.12. This issue is discussed in detail for soldier beams and lagging retaining wall systems in Section 3.2.11and in Section 4.2.10.3 for sheet-pile and wale retaining wall systems.

Description of these summaries (Table 5.1 through 5.14) are as follows:

a. The 25-ft-high soldier beam with timber lagging and post-tensioned tieback anchored wall system retaining loose granular soil with “safety with economy” design (Table 5.1).

b. The 25-ft-high soldier beam with timber lagging and post-tensioned tieback anchored wall system retaining loose granular soil with “stringent displacement control” design (Table 5.2).

c. The 35-ft-high soldier beam with timber lagging and post-tensioned tieback anchored wall system retaining loose granular soil with “safety with economy” design (Table 5.3).

d. The 35-ft-high soldier beam with timber lagging and post-tensioned tieback anchored wall system retaining loose granular soil with “stringent displacement control” design is shown in Table 5.4.

e. The 25-ft-high soldier beam with timber lagging and post-tensioned tieback anchored wall system retaining stiff cohesive soil with “safety with economy” design (Table 5.5).

f. The 25-ft-high soldier beam with timber lagging and post-tensioned tieback anchored wall system retaining stiff cohesive soil with “stringent displacement control” (Table 5.6).

g. The 35-ft-high soldier beam with timber lagging and post-tensioned tieback anchored wall system retaining stiff cohesive soil with “safety with economy” design (Table 5.7).

h. The 35-ft-high soldier beam with timber lagging and post-tensioned tieback anchored wall system retaining stiff cohesive soil with “stringent displacement control” design (Table 5.8).

i. The 25-ft-high vertical sheet pile with wales and post-tensioned tieback anchored wall system retaining stiff cohesive soil “safety with economy” design (Table 5.9).

j. The 25-ft-high vertical sheet piles with wales and post-tensioned tieback anchored wall system retaining stiff cohesive soil with “stringent displacement control” design (Table 5.10).

k. The 35-ft-high vertical sheet pile with wales and post-tensioned tieback anchored wall system retaining stiff cohesive soil “safety with economy” design (Table 5.11).

l. The 35-ft-high vertical sheet pile with wales and post-tensioned tieback anchored wall system retaining stiff cohesive soil “stringent displacement control” design (Table 5.12).

m. Summary of soldier beam design moments for the 25- and 35-ft-high drilled-in soldier beam with timber lagging and post-tensioned tieback anchored retaining walls (Table 5.13).

n. Summary of vertical sheet pile design moments for the 25- and 35-ft-high vertical sheet piles with wales and post-tensioned tieback anchored retaining walls (Table 5.14).

All solider beams sheet-piles and wales are Grade 50 steel.

Table 5.1
Summary of Results for Two-Tier, 25-ft Drilled-In Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Restraining Loose Granular Soils ("Safety with Economy" Design)

TWO-TIER DRILLED-IN SOLDIER BEAM DESIGN		
Wall height		25 ft
Soldier beam spacing		6 ft
Soldier beam design moment		41.5 kip-ft
Soldier beam size		2MC 8×20
Soldier beam length		31 ft
Drill shaft diameter		26 in.
Toe reaction		8.2 kips
Top-tier Anchor	H_1	6-ft 0-in.
	Anchor inclination	20°
	Design load	42.8 kips
	Unbonded length	14.65-ft < 15-ft, Use 15-ft minimum
	Bonded length*	12.2-ft
	Total length	28 ft
	Tendon size	Two 0.6-in. strands
Second-tier Anchor	H2	9-ft 6-in.
	Anchor inclination	20°
	Design load	45.5 kips
	Unbonded length	9.8-ft < 15-ft, Use 15-ft minimum
	Bonded length*	13 ft
	Total length	28 ft
	Tendon size	Two 0.6-in. strands

* Small-diameter straight-shaft gravity-grouted anchor.

Table 5.2
Summary of Results for Two-Tier, 25-ft Drilled-In Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Restraining Loose Granular Soils ("Stringent Displacement Control" Design)

TWO-TIER DRILLED-IN SOLDIER BEAM DESIGN		
Wall height		25 ft
Soldier beam spacing		6 ft
Soldier beam design moment		46.2 kip-ft
Soldier beam size		2MC 8x20
Soldier beam length		32 ft
Drill shaft diameter		26 in.
Toe reaction		9.1 kips
Top-tier Anchor	H_1	6-ft 0-in.
	Anchor inclination	20°
	Design load	47.6 kips
	Unbonded length	14.65-ft<15-ft, Use 15-ft minimum
	Bonded length*	13.6-ft
	Total length	29 ft
	Tendon size	Two 0.6-in. strands
Second-tier Anchor	H_2	9-ft 6-in.
	Anchor inclination	20°
	Design load	50.7 kips
	Unbonded length	9.8-ft < 15-ft, Use 15-ft minimum
	Bonded length*	14.5-ft
	Total length	30 ft
	Tendon size	Two 0.6-in. strands

* Small-diameter straight-shaft gravity-grouted anchor.

Table 5.3
Summary of Results for Three-Tier, 35-ft Drilled-in Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Restraining Loose Granular Soils ("Safety with Economy" Design)

THREE-TIER DRILLED-IN SOLDIER BEAM DESIGN		
Wall height		35 ft
Soldier beam spacing		6 ft
Soldier beam design moment		54.4 kip-ft
Soldier beam size		2 MC 8x20
Soldier beam length		44 ft
Drill shaft diameter		26 in.
Toe reaction		10.7 kips
Top-tier Anchor	H_1	6-ft 6-in.
	Anchor inclination	20°
	Design load	58.2 kips
	Unbonded length	21.46-ft
	Bonded length*	16.6-ft
	Total length	39 ft
	Tendon size	Two 0.6-in. strands
Second-tier Anchor	H_2	9-ft 6-in.
	Anchor inclination	20°
	Design load	60.9 kips
	Unbonded length	16.64-ft
	Bonded length*	17.4-ft
	Total length	35 ft
	Tendon size	Two 0.6-in. strands
Third-tier Anchor	H_3	9-ft 6-in.
	Anchor inclination	20°
	Design load	59.6 kips
	Unbonded length	11.82-ft < 15-ft, Use 15-ft minimum
	Bonded length*	17-ft
	Total length	32 ft
	Tendon size	Two 0.6-in. strands

* Small-diameter straight-shaft gravity-grouted anchor.

Table 5.4
Summary of Results for Three-Tier, 35-ft Drilled-in Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Restraining Loose Granular Soils ("Stringent Displacement Control" Design)

THREE-TIER DRILLED-IN SOLDIER BEAM DESIGN		
Wall height		35 ft
Soldier beam spacing		6 ft
Soldier beam design moment		60.6 kip-ft
Soldier beam size		2 MC 8×22.8
Soldier beam length		44 ft
Drill shaft diameter		26 in.
Toe reaction		12 kips
Top-tier Anchor	H_1	6-ft 6-in.
	Anchor inclination	20°
	Design load	64.7 kips
	Unbonded length	21.46-ft
	Bonded length*	18.5-ft
	Total length	40 ft
	Tendon size	Two 0.6-in. strands
Second-tier Anchor	H_2	9-ft 6-in.
	Anchor inclination	20°
	Design load	67.6 kips
	Unbonded length	16.64-ft
	Bonded length*	19.3-ft
	Total length	36 ft
	Tendon size	Two 0.6-in. strands
Third- tier Anchor	H_3	9-ft 6-in.
	Anchor inclination	20°
	Design load	66.2 kips
	Unbonded length	11.82-ft < 15-ft, Use 15-ft minimum
	Bonded length*	18.9-ft
	Total length	34 ft
	Tendon size	Two 0.6-in. strands

* Small-diameter straight-shaft gravity-grouted anchor.

Table 5.5
Summary of Results for Two-Tier, 25-ft Drilled-in Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Restraining Stiff Cohesive Soil ("Safety with Economy" Design)

TWO-TIER DRILLED-IN SOLDIER BEAM DESIGN		
Wall height		25 ft
Soldier beam spacing		6 ft
Soldier beam design moment		38.7 kip-ft
Soldier beam size		2 MC 8×20
Soldier beam length		28 ft
Drill shaft diameter		26 in.
Toe reaction		7.6 kips
Top-tier Anchor	H_1	6-ft 0-in.
	Anchor inclination	20°
	Design load	40 kips
	Unbonded length	19.8 ft
	Bonded length*	30.3-ft
	Total length	51 ft
	Tendon size	Two 0.6-in. strands
Second-tier Anchor	H_2	9-ft 6-in.
	Anchor inclination	20°
	Design load	42.5 kips
	Unbonded length	12.3-ft < 15-ft, Use 15-ft minimum
	Bonded length*	32.2-ft
	Total length	48 ft
	Tendon size	Two 0.6-in. strands

* 12-in.-diameter straight-shaft gravity-grouted anchor.

Table 5.6
Summary of Results for Two-Tier, 25-ft Drilled-in Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Restraining Stiff Cohesive Soil ("Stringent Displacement Control" Design)

TWO-TIER DRILLED-IN SOLDIER BEAM DESIGN		
Wall height		25 ft
Soldier beam spacing		6 ft
Soldier beam design moment		44.4 kip-ft
Soldier beam size		2 MC 8×20
Soldier beam length		28-ft
Drill shaft diameter		26 in.
Toe reaction		8.8 kips
Top-tier Anchor	H_1	6-ft 0-in.
	Anchor inclination	20°
	Design load	45.8 kips
	Unbonded length	19.8 ft
	Bonded length*	34.7 ft
	Total length	55 ft
	Tendon size	Two 0.6-in. strands
Second-tier Anchor	H_2	9-ft 6-in.
	Anchor inclination	20°
	Design load	48.7 kips
	Unbonded length	12.3-ft < 15-ft, Use 15-ft minimum
	Bonded length*	36.9-ft
	Total length	52 ft
	Tendon size	Two 0.6-in. strands

* 12-in.-diameter straight-shaft gravity-grouted anchor.

Table 5.7
Summary of Results for Three-Tier, 35-ft Drilled-in Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Restraining Stiff Cohesive Soil (“Safety with Economy” Design)

THREE-TIER DRILLED-IN SOLDIER BEAM DESIGN		
Wall height		35 ft
Soldier beam spacing		6 ft
Soldier beam design moment		50.8 kip-ft
Soldier beam size		2 MC8×20
Soldier beam length		39 ft
Drill shaft diameter		26 in.
Toe reaction		10 kips
Top-tier Anchor	H_1	6-ft 6-in.
	Anchor inclination	20°
	Design load	54.3 kips
	Unbonded length	29.24-ft
	Bonded length*	40-ft
	Total length	70 ft
	Tendon size	Two 0.6-in. strands
Second-tier Anchor	H_2	9-ft 6-in
	Anchor inclination	20°
	Design load	56.9 kips
	Unbonded length	21.82-ft
	Bonded length*	40-ft
	Total length	62-ft
	Tendon size	Two 0.6-in. strands
Third-tier Anchor	H_3	9-ft 6-in.
	Anchor inclination	20°
	Design load	55.7 kips
	Unbonded length	14.41-ft < 15-ft, Use 15-ft minimum
	Bonded length*	40-ft
	Total length	55 ft
	Tendon size	Two 0.6-in. strands

* Post-grouted (regroutable) ground anchor.

Table 5.8
Summary of Results for Three-Tier, 35-ft Drilled-in Soldier Beam with Timber Lagging and Post-Tensioned Tieback Anchored Wall System Restraining Stiff Cohesive Soil ("Stringent Displacement Control" Design)

THREE-TIER DRILLED-IN SOLDIER BEAM DESIGN		
Wall height		35 ft
Soldier beam spacing		6 ft
Soldier beam design moment		55.4 kip-ft
Soldier beam size		2 MC 8×20
Soldier beam length		39 ft
Drill shaft diameter		26 in.
Toe reaction		11.5 kips
Top-tier Anchor	H_1	7-ft 3-in.
	Anchor inclination	20°
	Design load	62.3 kips
	Unbonded length	28.65-ft
	Bonded length*	40-ft
	Total length	69 ft
	Tendon size	Two 0.6-in. strands
Second-tier Anchor	H_2	9-ft 3-in
	Anchor inclination	20°
	Design load	65.1 kips
	Unbonded length	21.43 ft
	Bonded length*	40-ft
	Total length	62 ft
	Tendon size	Two 0.6-in. strands
Third-tier Anchor	H_3	9-ft 3-in.
	Anchor inclination	20°
	Design load	63.8 kips
	Unbonded length	14.22-ft < 15-ft, Use 15-ft minimum
	Bonded length*	40-ft
	Total length	55 ft
	Tendon size	Two 0.6-in. strands

* Post-grouted (regroutable) ground anchor.

Table 5.9
Summary of Results for Two-Tier, 25-ft-High Vertical Sheet Piles with Wales and Post-Tensioned Tieback Anchored Wall System Restraining Stiff Cohesive Soil ("Safety with Economy" Design)

TWO-TIER SHEET-PILE AND WALE DESIGN		
Wall height		25 ft
Sheet-pile design moment		6.5 ft-kip/ft
Sheet-pile size		AZ 13
Wales size		2C 9×20
Toe embedment length		6-ft
Sheet-pile length		31 ft
Toe reaction		1.28 kip/ft
Horizontal anchor spacing		8.8 ft
Top-tier Anchor	H_1	6-ft 0-in.
	Anchor inclination	20°
	Design load	58.6 kips
	Unbonded length	19.8 ft
	Bonded length*	40-ft
	Total length	60 ft
	Tendon size	Two 0.6-in. strands
Second-tier Anchor	H_2	9-ft 6-in.
	Anchor inclination	20°
	Design load	62.3 kips
	Unbonded length	12.3-ft < 15-ft, Use 15-ft minimum
	Bonded length*	40-ft
	Total length	55 ft
	Tendon size	Two 0.6-in. strands
* Post-grouted (regroutable) ground anchor.		

Table 5.10
Summary of Results for Two-Tier, 25-ft-High Vertical Sheet Piles with Wales and Post-Tensioned Tieback Anchored Wall System Restraining Stiff Cohesive Soil ("Stringent Displacement Control" Design)

TWO-TIER SHEET-PILE AND WALE DESIGN		
Wall height		25 ft
Sheet-pile design moment		7.4 ft-kip/ft
Sheet-pile size		AZ 13
Wales size		2C 10×20
Toe embedment length		6-ft
Sheet-pile length		31 ft
Toe reaction		1.46 kip/ft
Horizontal anchor spacing		8.8-ft
Top-tier Anchor	H_1	6-ft 0-in.
	Anchor inclination	20°
	Design load	67.2 kips
	Unbonded length	19.8 ft
	Bonded length*	40 ft
	Total length	60 ft
	Tendon size	Three 0.6-in. strands
Second-tier Anchor	H_2	9-ft 6-in.
	Anchor inclination	20°
	Design load	71.4 kips
	Unbonded length	12.3-ft < 15-ft, Use 15-ft minimum
	Bonded length*	40 ft
	Total length	55 ft
	Tendon size	Three 0.6-in. strands

* Post-grouted (regroutable) ground anchor.

Table 5.11
Summary of Results for Three-Tier, 35-ft-High Vertical Sheet Piles with Wales and Post-tensioned Tieback Anchored Wall System Restraining Stiff Cohesive Soil (“Safety with Economy” Design)

THREE-TIER SHEET-PILE AND WALE DESIGN		
Wall height		35 ft
Sheet-pile design moment		8.5 ft-kip/ft
Sheet-pile size		AZ 13
Wales size		2C 10×25
Toe embedment length		7-ft
Sheet-pile length		42 ft
Toe reaction		1.67 kip/ft
Horizontal anchor spacing		8.8 ft
Top-tier Anchor	H_1	6-ft 6-in.
	Anchor inclination	20°
	Design load	79.7 kips
	Unbonded length	29.24-ft
	Bonded length*	40-ft
	Total length	70 ft
	Tendon size	Three 0.6-in. strands
Second-tier Anchor	H_2	9-ft 6-in
	Anchor inclination	20°
	Design load	83.4 kips
	Unbonded length	21.82-ft
	Bonded length*	40-ft
	Total length	62 ft
	Tendon size	Three 0.6-in. strands
Third-tier Anchor	H_3	9-ft 6-in
	Anchor inclination	20°
	Design load	81.7 kips
	Unbonded length	14.41-ft < 15-ft, Use 15-ft minimum
	Bonded length*	40 ft
	Total length	55 ft
	Tendon size	Three 0.6-in. strands

* Post-grouted (regroutable) ground anchor.

Table 5.12
Summary of Results for Three-Tier, 35-ft-High Vertical Sheet Piles with Wales and Post-Tensioned Tieback Anchored Wall System Restraining Stiff Cohesive Soil ("Stringent Displacement Control" Design)

THREE-TIER SHEET-PILE AND WALE DESIGN		
Wall height		35 ft
Sheet-pile design moment		9.2 ft-kip/ft
Sheet-pile size		AZ 13
Wales size		2C10×30
Toe embedment length		7-ft
Sheet-pile length		42 ft
Toe reaction		1.91 kip/ft
Horizontal anchor spacing		8.8-ft
Top-tier Anchor	H_1	7-ft 3-in.
	Anchor inclination	20°
	Design load	91.3 kips
	Unbonded length	28.65-ft
	Bonded length*	40-ft
	Total length	69 ft
	Tendon size	Three 0.6-in. strands
Second-tier Anchor	H_2	9-ft 3-in
	Anchor inclination	20°
	Design load	95.5 kips
	Unbonded length	21.43-ft
	Bonded length	40-ft
	Total length	62-ft
	Tendon size	Three 0.6-in. strands
Third-tier Anchor	H_3	9-ft 3-in.
	Anchor inclination	20°
	Design load	93.6 kips
	Unbonded length	14.22-ft
	Bonded length*	40-ft
	Total length	55-ft
	Tendon size	Three 0.6-in. strands

* Post-grouted (regroutable) ground anchor.

Table 5.13
Summary of Soldier Beam Design Moments for the 25-ft- and 35-ft-High Drilled-in Soldier Beam with Timber Lagging and Post-tensioned Tieback Anchored Retaining Walls

Tieback Wall Height (ft)	Soil Type	Number Rows of Anchors	Soldier Beam and Anchor Spacing (in-plan) (ft)	Soldier Beam Design Moment (ft-kips)		*Increase (%)
				Safety with Economy	Stringent Displacement Control	
25	Granular	2	6	41.5	46.2	11
35	Granular	3	6	54.4	60.6	11
25	Cohesive	2	6	38.7	44.4	15
35	Cohesive	3	6	50.8	55.4	9

* Increase (in percent) of "stringent displacement control" moment demand to "safety with economy" moment demand.

Table 5.14
Summary of Vertical Sheet-Pile Design Moments for the 25-ft- and 35-ft-High Vertical Sheet Piles with Wales and Post-tensioned Tieback Anchored Retaining Walls

Tieback Wall Height (ft)	Soil Type	Number Rows of Anchors	Anchor Spacing (in-plan) (ft)	Sheet-Pile Design Moment (ft-kips/ft run of wall)		*Increase (%)
				Safety with Economy	Stringent Displacement Control	
25	Cohesive	2	8.8	6.5	7.4	14
35	Cohesive	3	8.8	8.5	9.2	8

* Increase (in percent) of "stringent displacement control" moment demand to "safety with economy" moment demand.

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Appendix A
Drained Shear Strength Parameters for Stiff Clay Sites

A.1 Introduction

Although permanent ground anchor walls are seldom constructed in normally consolidated clay deposits, they are routinely built in overconsolidated clays. The apparent earth pressure design approach for tieback walls constructed at stiff clay sites for undrained (short-term) and drained (long-term) conditions is described in FHWA-RD-97-130 and in Strom and Ebeling (2001). The development of R-y curves for stiff clay sites by the reference deflection method is described in FHWA-RD-98-066. This appendix presents information required to develop the drained shear strength parameters (i.e., drained friction angle) for overconsolidated clays, since the drained friction angle for a normally consolidated clay and intact overconsolidated clay are not the same. This information is taken from FHWA-RD-97-130 and is presented to facilitate the development of earth pressures and R-y curves for use in the construction-sequencing analyses illustrated in the main text of this report. Terms used in describing and developing drained shear strength parameters for stiff clay sites are as follows:

s = drained shear strength

$\sigma^{'}$ = effective normal stress

$\phi^{'}$ = drained friction angle

c = cohesion intercept

OCR = overconsolidation ratio

m = factor defining the extent of fissures in the soil

A.2 Drained Shear Strength of Overconsolidated Clay (FHWA-RD-97-130)

The drained strength of a normally consolidated cohesive soil depends on the drained friction angle ($\phi^{'}$) and the effective normal stress ($\sigma^{'}$) and is expressed by the relationship

$$s = \sigma^{'} \tan \phi^{'} \tag{A.1}$$

The effective normal stress ($\sigma^{'}$) on the shear plane is the total normal stress on the plane less the pore-water pressure after equilibrium is reached. Friction angle ($\phi^{'}$) depends on the clay content of the soil, clay mineralogy, and arrangement of clay particles. Figure A.1 (from Terzaghi, Peck, and Mesri 1996) shows how $\phi^{'}$ varies with the plasticity index for normally consolidated clays.

Data points far above the line represent soils that have an effective normal stress less than 1,000 psf and a clay content less than 20 percent, and data points well below the line represent soils having effective normal stresses greater than 8,350 psf and clay contents greater than 50 percent.

The drained shear strength of overconsolidated clay should be greater than the drained shear strength of a similar soil in a normally consolidated state. The drained shear strength of saturated overconsolidated clay is called the intact shear strength, and is defined with respect to the cohesion intercept ($c^{'}$) and the friction angle ($\phi^{'}$) of a Mohr failure envelope by Equation A.2.

$$s = c^{'} + \sigma^{'} \tan \phi^{'} \tag{A.2}$$

Friction angles for the intact overconsolidated clay are higher at effective stresses lower than the preconsolidation pressure, and trend toward the normally consolidated friction angle at high effective normal stresses. Terzaghi, Peck, and Mesri (1996) used Equation A.3 to express the drained strength of overconsolidated clay in terms of the drained strength of the same soil in its normally consolidated state, the overconsolidation ratio (OCR), and term *m*, which depends on the fissures in the soil.

$$s = \sigma^{'} \tan \phi^{'} OCR^{1-m} \tag{A.3}$$

The preconsolidation pressure used to determine the OCR in Equation A.2 is the effective normal stress where the Mohr diagram failure envelope for the overconsolidated clay joins the failure envelope for the normally consolidated clay. The exponent *m* for clays and shales is given in Table A-1.

Terzaghi, Peck, and Mesri (1996) defined intact soils as soils that are undisturbed and unfissured, and destructured soils as slightly fissured stiff clays and shales and soft clays sheared to a large-strain condition. Destructured soils are stronger than fully strained softened stiff clays or shales or completely remolded soft clays. Fully strained softened or remolded clays will have an *m* of 1 (approximately), and their drained shear strength will approximately equal the normally consolidated shear strength.

Table A.1
Values of *m* in Equation A.3

Soil Description	m	
	Intact Soil	Destructured Soil
Stiff clays and shales	0.5 – 0.6	0.6 – 0.8
Soft clays	0.6 – 0.7	0.6 – 0.9

Source: Terzaghi, Peck, and Mesri (1996).

Drained shear strength of heavily overconsolidated clay depends upon the condition of the clay after unloading and swelling. The drained shear strength of a badly fissured and jointed clay may be reduced to its fully softened shear strength (strength in its normally consolidated state). If large displacements have occurred within heavily overconsolidated stiff clay in the geologic past, the drained friction angle may be reduced to a residual value along planes where the displacements occurred. These planes must be continuous for a considerable distance for the shear strength to be reduced to a residual value. The residual friction angle is equal to or lower than the drained friction angle of a normally consolidated clay (fully strain softened). When the displacements occur, the clay particles are reoriented parallel to the direction of shearing. The magnitude of the friction angle reduction depends upon the clay content and the shape of the clay particles. The residual friction angle will be low for soils that have a high percentage of plate-shaped clay minerals. For an anchored wall, residual shear strength is mobilized only when displacements occur along pre-existing shear surfaces. These surfaces have to be oriented in a direction that will affect the stability of the anchored wall, or the behavior of the wall will not be dependent upon the residual shear strength of the soil. Figure A.2 (from Patton and Henderson 1974) gives drained residual friction angles for rock gouge material as a function of plasticity index.

Terzaghi, Peck, and Mesri (1996) present the residual friction angle as a function of the friction angle of normally consolidated clays (Figure A.3).

Both figures illustrate the strength reduction that can occur when a stiff, heavily overconsolidated clay is sheared, reducing the strength to a residual value.

Figure A.4 combines previously described relationships and serves as a guide for estimating the drained friction angle for fine-grained soils in different states of stress or disturbance. The line representing the normally consolidated state is the trend line from Figure A.1. Lines representing the overconsolidated soils were determined by setting Equation A.1 equal to Equation A.3 and solving for ϕ' in Equation A.1. Values selected for *m* in Equation A.3 are presented in Figure A.4. Curves representing intact and destructured soils were drawn for clays with an OCR of 2. The range for the residual friction angles was developed from Figures A.2 and A.3.

It should be noted that the short-term (undrained) apparent earth pressures could be greater than the pressures computed using the drained shear strength parameters.

Atterberg limits for the clay, the OCR, the extent of fissuring, and the nature and orientation of joints or shears are needed to use Figure A.4 for estimating the drained friction angle. After estimating the drained friction angle, one should determine the earth pressures associated with the drained condition and the pore-water pressures, and compare them with the earth pressures associated with the undrained shear strength. The pressures that give the greatest demands with respect to the tieback wall structural component of interest should be used for design of that component. Demands associated with the undrained earth pressure condition may be greater than those associated with drained earth pressures plus water pressure. When the wall is going to be built in a heavily overconsolidated deposit, local experience should guide in determining the degree of disturbance and the soil strength. Laboratory tests can be used to determine drained shear strength parameters, but tests done on samples recovered from the deposit may not accurately represent the strength of a fissured soil. In addition to testing, local experience, and understanding of the geologic events that have affected soils at the site, the relationships in Figure A.3 should be considered when estimating the drained friction angle.

Stress relief in heavily overconsolidated fine-grained soils may result in a strength reduction. How this reduction affects anchored walls is not clear. Sills, Burland, and Czechowski (1977) reported that stress relief in a 26-ft-deep excavation in London clay resulted in deep-seated movements behind ground anchors that were twice the height of the wall but no increase in anchor load. If there is a concern that wall movements will cause stress relief in the ground, the measured drained strength can be reduced. If stress relief occurs, the strengths will likely be greater than the normally consolidated drained shear strength (see Figure A.3). Drained shear strengths should not be reduced below the normally consolidated strengths unless deposit has been sheared in the geologic past and the discontinuities are oriented in a direction that affects the stability of the wall.

Poor drilling techniques using air or water to clean the drill hole may fracture the soil and reduce the soil's shear strength or pressurize the drilling fluid in open fractures. The strength reduction or the effect of pressurizing the drilling fluid is not considered in the design. Fracturing the ground is controlled by preventing collaring of the hole when drilling with air or water. A collar occurs when the hole becomes blocked and cuttings no longer return up the drill hole to the surface. If a collar occurs, the pressurized drilling fluid (air or water) is forced into the ground, disrupting the formation. Auger drilling methods will not disrupt the soil where collaring is likely.

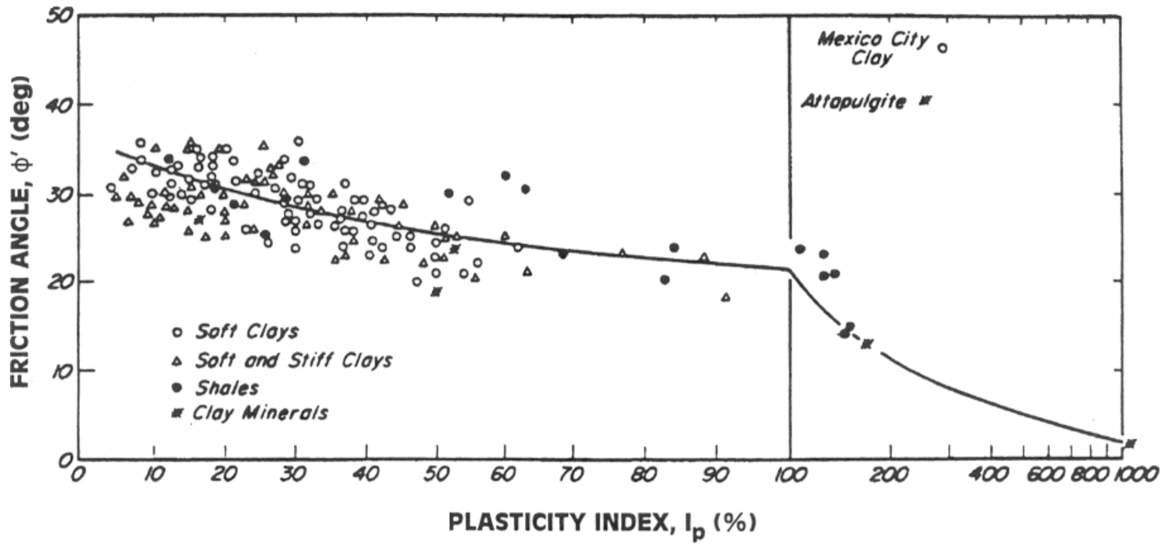

Figure A.1. Undrained friction angle ϕ' for normally consolidated clays in terms of plasticity index (after Figure 35, FHWA-RD-97-130)

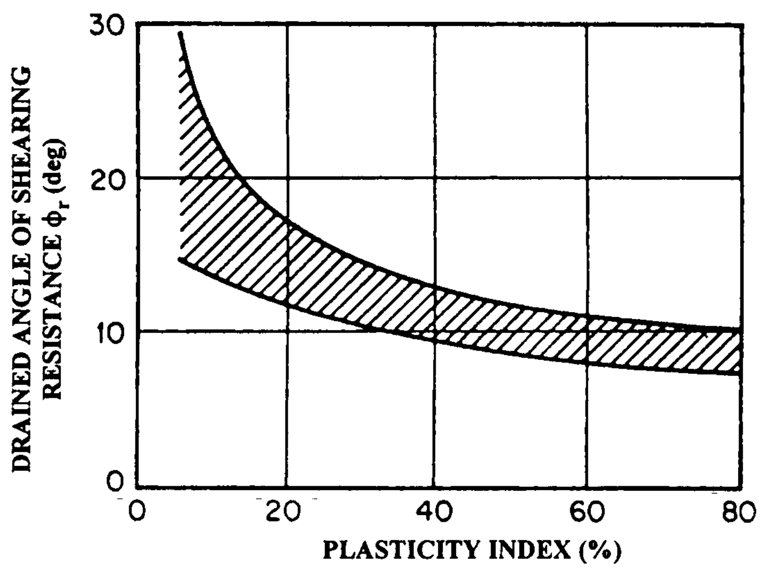

Figure A.2. Approximate relationship between the drained residual friction angle and plasticity index for rock gouge (after Figure 36, FHWA-RD-97-130)

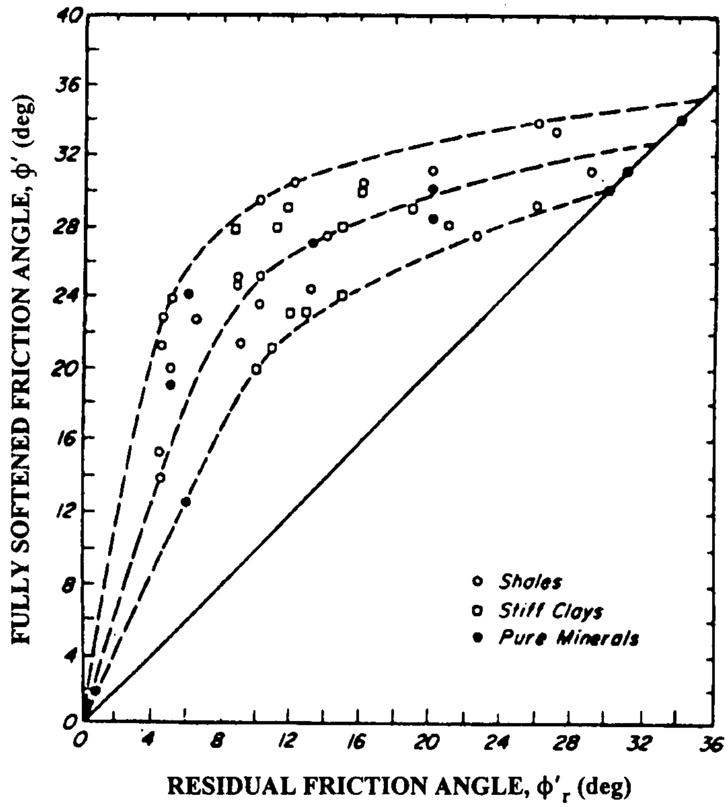

Figure A.3 Relationship between fully softened ϕ' and residual ϕ' (after Figure 37, FHWA-RD-97-130)

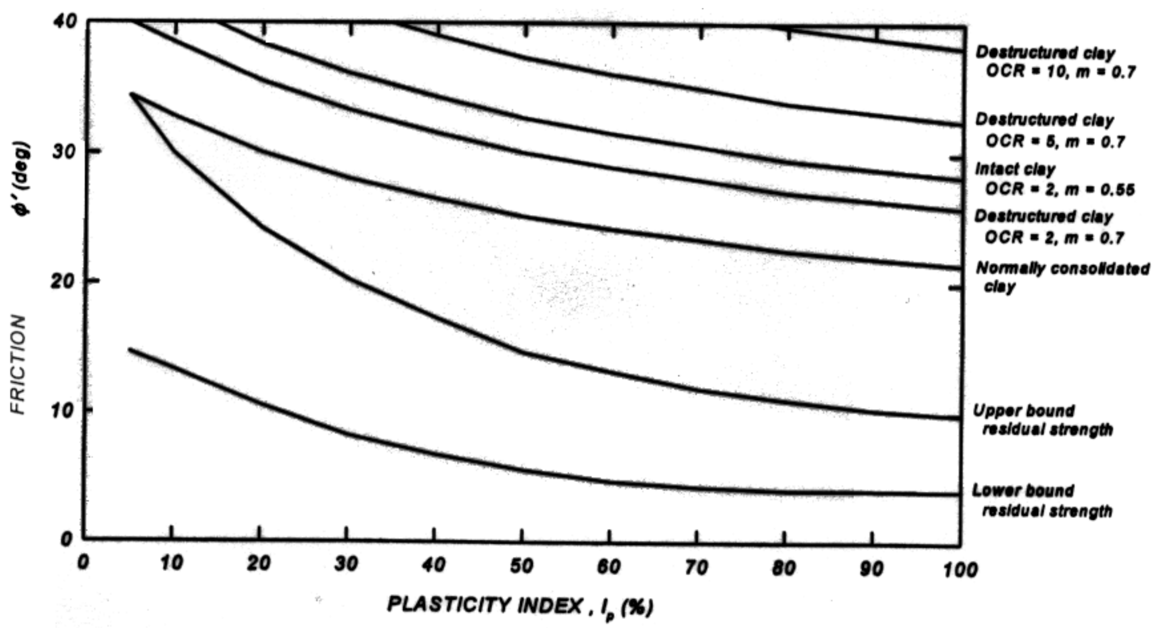

Figure A.4. Friction angle ϕ' for clays in different states as a function of plasticity index (after Figure 37, FHWA-RD-97-130)

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14. ABSTRACT
In practice, the procedures used to design flexible tieback wall systems differ from those used to design stiff tieback wall systems. In the design of flexible tieback wall systems(with post-tensioned tieback anchors), apparent pressure diagrams are commonly used to represent the maximum loads the tieback wall system might experience during construction. Apparent pressure diagrams used in an equivalent beam on rigid supports analysis are demonstrated in this report. Analyses are performed for flexible wall systems in both cohesionless and clay soil. Flexible wall systems include a soldier beam–wood lagging system and a sheet-pile and wale system. Wall heights of 25, 35, and 50 ft (8, 11, and 15 m) are evaluated, all with a horizontal retained soil surface.

Apparent pressures are developed on a "total load" approach using limiting equilibrium procedures. Apparent pressure diagrams are nonsymmetrical in shape, as recommended in FHWA-RD-97-130 ("Design Manual for Permanent Ground Anchor Walls," Federal Highway Administration).

Designs are provided for two performance objectives: "safety with economy" and "stringent displacement control." A factor of safety of 1.3 is used for the safety with economy designs for which displacement control is not a significant concern. A factor of safety of 1.5

(Continued)

15. SUBJECT TERMS

Apparent pressure diagram	Flexible wall	Soldier beam
Equivalent beam	Ground anchor wall	Tieback wall
	Sheet-pile system	Timber lagging

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is used for the stringent displacement control designs, for which it is assumed that displacements must be minimized to prevent settlement-related damage to nearby structures.

Comparisons are made between the safety with economy and the stringent displacement control designs for the wall heights indicated above.